STATE-OF-THE-ART ASSESSMENT ON DRILLED AND GROUTED PILES FOR TENSION LEG PLATFORM FOUNDATIONS

Prepared by

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February 12, 1987

Sandia National Laboratories
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Attention: Mr. Dennis Engi

Re: State-of-the-Art Assessment on Drilled and Grouted Piles for Tension Leg Platform Foundations
Contract #50-0744 - Document #04-7710
ETC Project No. 86-004

Gentlemen:

We are pleased to submit ten (10) copies and one (1) reproducible original of our report entitled "State-of-the-Art Assessment on Drilled and Grouted Piles for Tension Leg Platform Foundations." The work was performed under the terms of Document No. 04-7710 with Sandia National Laboratories (SNL) dated March 12, 1986. This project was carried out as an extension of a previous study in 1983 performed by The Earth Technology Corporation for SNL under Contract #50-0744 entitled "State-of-the-Art Review in Foundation Design and Analysis for Tension Leg Platforms and Vertically Moored Platforms."

The enclosed report summarizes the present state-of-the-art in designing, fabricating, installing and testing drilled and grouted piles to anchor foundations for tension leg platforms (TLP). The work gives an overview of the industry experience and presents the drilling, grouting and design technology presently available. This report also discusses the major technology gaps which are apparent and their proposed solutions.

We have enjoyed once again to work on this project and look forward to further projects with Sandia.

Very truly yours,

Scott R. Bamford
Senior Project Engineer

copies: 10

cc: Mr. Jean Audibert, ETC
Mr. Charles Smith, MMS

EXECUTIVE SUMMARY

General

This report presents the current state of knowledge and technology on drilled and grouted piles in relation to their applications to deep water tension leg platforms (TLP). The work was sponsored by the Minerals Management Service (MMS) through Sandia National Laboratories (SNL). The material contained in this work was gathered from different sources which include literature surveys, experience of the project team, and experience from oil and gas industry representatives, particularly on offshore drilling and grouting operations. Information on onshore drilled shafts and piers, as well as experience from oil well casing installation were included since the associated technologies are similar to those used for constructing drilled and grouted piles and also because data on drilled and grouted piles is very scarce and must be complemented by other available information.

With the increasing demand for oil and gas, offshore production facilities have been moving into deeper waters. Offshore drilling has been carried out in water depths exceeding 6,000 ft of the U.S. east coast with future wells planned to be drilled in 7,500 ft water depth in the Gulf of Mexico. The technology to drill deep water oil wells seem to be far ahead of the existing technology to install structures and drilling facilities at such sites.

Conventional fixed jacket platforms have been widely used worldwide due to their excellent record. However, as the water depth exceeds about 1,500 ft, fixed jacket platforms become economically and technically unattractive. Several investigators are considering compliant structures such as buoyant towers, guyed towers and TLPs. The latter typically consists of a floating structure (which contains drilling and production facilities) anchored to the seafloor structures through pre-tensioned tendons attached to seafloor anchoring templates. These seafloor structures, which may also function as a template for drilling, are usually supported or anchored to the seafloor by pile foundations.

Pile Foundations

Driven piles are most often used offshore. Thus a great deal of information is available regarding the design, installation and performance of such foundation members. Successful performance in the past has boosted the confidence level in the use of driven piles more than for any other type of foundation.

However, as the water depth increases, difficulties involved with pile driving operations escalate. The major stumbling block is the present lack of underwater hammers capable of operating in water depths in excess of 1,500 ft. Drilled and grouted piles are therefore being considered by industry to anchor TLP foundations at deep water sites. Deep water drilling and grouting technology is presently available for oil well casing installation. This can be used to install drilled and grouted piles. Drilled and grouted piles may also be more attractive for TLP applications in that they alleviate inherent effects from pile driving such as cavity expansion, thixotropic effects, and consolidation.

Drilled and grouted piles basically consist of a steel pipe pile (insert) grouted in a predrilled borehole. The construction operation often starts with the installation of a surface casing to prevent the formation of a surface crater and possible sloughing of the soft surficial soils. Drilling of the borehole then proceeds with the use of drilling fluids to stabilize the borehole. The insert pile is then run into the borehole and grouted in place in one or several stages. The latter is needed when hydraulic fracturing would occur under the grout pressure if only one stage was used.

TLP Loading Characteristics

Constant buoyancy forces, in conjunction with environmental loading (wind, waves, currents) on the floating structure, impart to the tendon and the anchoring pile foundation an unusual loading condition consisting of a constant tension with an additional cyclic load component.

Understanding of the behavior of piles under tensile cyclic loads is therefore necessary for the design of TLP foundation piles. Moreover, the effects from the installation operations (e.g. drilling, grouting) on the pile behavior need to be determined. Our assessment of the state-of-the-art on this subject was thus divided into four aspects: (1) industry's experience, (2) drilling technology, (3) grouting technology, and (4) analysis and design methods.

Industry's Experience

Because driven piles have been used almost exclusively offshore, the industry's experience with offshore drilled and grouted pile is very limited. Drilled and grouted piles have been used offshore only when pile driving was not possible (e.g. in strong soils) or when detrimental to the load carrying capacity (e.g. in calcareous sediments). However, these piles were usually installed in water depths less than 500 ft. Drilled and grouted piles have also been used as mooring anchors for drillships, but were usually small and short as compared to the piles that would be required to anchor a TLP in deep waters.

Installation of drilled and grouted piles is similar to installation of oil well casings in many respects. Oil well casings have been installed in nearly 7,000 ft of water. The equipment is therefore readily available to install drilled and grouted piles. However, practical problems from drilling, grouting and quality control procedure still exist as experienced from the installation of drilled and grouted piles for the Thistle A and Piper jackets in the North Sea. These problems include:

1) Hydraulic fracture of soil formations (due to our pressure from the drilling fluids or grout) leading to excessive loss of drilling fluids and grout

2) Instability of the borehole

3) Excessive borehole diameter and grout volume

4) Excessive installation time

Public domain information on the behavior of drilled and grouted piles under cyclic tension loading is practically nonexistent at present. However, experience with similar foundations on-land and near-shore can serve as a valuable data base in understanding the behavior of drilled and grouted piles. As an example, high-pressure multiple grouting was found to improve significantly the holding capacity of soil anchors. Drilled and grouted piles would also have similar benefits from this technique by either increasing the load carrying capacity or remedying the problems which occurred during installation.

Drilling Technology

As mentioned previously, the drilling technology in deep water has surpassed any other aspects of drilled and grouted piles. Drilling equipment and vessels are available with a capability to drill a suitable size borehole (i.e., 26 to 54-in diameter) in deep waters (i.e., 1,000 to 4,000 ft) to the penetration depth required for drilled and grouted piles (i.e., 300 to 600 ft). The drilling capability to obtain soil or rock samples for site characterization is even greater (up to 18,000 ft water depth and 2,850 ft penetration).

As previously mentioned, a surface casing is generally required to prevent surface cratering and sloughing of the weak surficial soils. Several methods exist to install a surface casing which includes: (1) fixed jetting assembly, (2) drilled-in conductor, (3) turbo-drill, (4) jetting with internal return, (5) expendable casing drill, (6) suction method, and (7) vibratory technique. Each of these techniques has its specific advantages and drawbacks. Selection of one technique over another will result from site specific considerations such as soil conditions, water depth, available equipment and local experience.

There are a number of concerns which require further study to improve the reliability of drilling methods for drilled and grouted piles. These include:

1) Surface casing installation procedure and potential problems
2) Borehole stability
3) Hydraulic fracture
4) Mudcake formation
5) Soil disturbance
6) Hole verticality
7) Quality control

Grouting Technology

Grouting techniques vary according to their specific applications. The technique used in grouting oil well casings can be applied to drilled and grouted piles. Grouting techniques available at present include initial or primary grouting and high-pressure multiple-injection grouting. Conventional primary grouting through a bottom float shoe assembly uses only gravity to allow grout to fill in the annulus between the insert pile and the borehole wall. Hydraulic fracture in the soil formation may occur if the grout column is too high and the soils have low shear strengths. Therefore, primary grouting is sometimes done in successive stages.

There are several techniques for primary grouting including (1) inner string method, (2) grout line method, and (3) delayed set method. The inner string method seems to be more suitable for drilled and grouted piles. It uses a single retrievable grout line assembly inside the pile with a sealing adapter (float shoe) at the pile tip. Multiple stage grouting can be done by using ball or plug-operated diverter valves placed in the grout line.

High-pressure multiple-injection grouting (HPMIG) is a method developed by Soletanche of France. The technique involves the successive use of high-pressure injections to fracture previous layers of grout in order to a) build up a larger grout bulb, b) possibly reconsolidate the surrounding soils or c) impregnate the soil with grout. These effects solely or in combination have been shown to increase the load carrying capacity of drilled and grouted piles. This method has recently been used to regrout the bottom portion of a drilled and grouted instrumented test pile in the Gulf of Mexico. It has also been applied for the re-installation of a well casing in unstable permafrost soils offshore. Although this technique has not been used extensively offshore, it warrants further consideration in view of its unique features and advantages.

Various grout formulations and additives exist to suit different applications. The grout formulations are classified in relation to their density, setting time, and final strength. Additives are sometimes used to control the properties of grout such as setting time, viscosity, volume change characteristics. The choice of grout type to suit a particular drilled and grouted pile application would depend on such factors as soil types, loading conditions, availability, method of grouting and cost.

Quality control procedures in the grouting operation can be classified into those performed during or immediately after grouting and those performed after the grout has set. During grouting, density and grout levels can be monitored using such tools as radioactive densometer, temperature or resistivity sensors and pre-installed grout pressure sensors. The latter may also be used to monitor the grout pressure in order to avoid hydraulic fracturing.

Quality control measures after the grout has set include strength tests of grout samples and running of logging tools to obtain information on grout geometry and interface bond quality, if possible. There are several types of logging tools available at present. Most of them were originally developed for use for oil well casing to check integrity of grout annulus and quality of the steel-cement bond. For drilled and grouted piles, some of these tools may need recalibration or modifications in order to yield useful information regarding bond and grout quality, grout thickness and contact between grout and soil. Extensive research in these area is very important since the capacity of drilled and grouted piles depends on the grout strength and geometry as well as the interface bond strengths.

Analysis and Design of Drilled and Grouted Piles

The behavior of drilled and grouted piles under cyclic tension loading is complex and not well understood. One important unknown is the behavior of the composite pile cross section made of two materials (cement and steel) with drastically different behavior under tension. The effects from installation procedures and the uncertainty regarding the geometry of the piles and the quality of the bonds at the steel-grout and grout-soil interfaces make the problems even more indeterminate.

There are only a few documented load test data on offshore drilled and grouted piles and those were performed in soil conditions (e.g. calcareous soils) which may not be of immediate interest to TLP sites. The information, discussions and recommendations presented in this report were thus by necessity mostly related to data of compressive load tests on drilled shafts, piers, bored piles, and soil anchors. Design procedures for driven piles were also included whenever applicable or when they represented the only guidance available.

General procedures for the design of drilled and grouted piles are difficult to establish because so many effects control the load carrying capacity of the piles. One of the major factors is related to the installation effects which determine the overall quality of the pile e.g. the stiffness, steel-grout bond strength, grout integrity and skin friction. Until enough load test data are documented together with further research to fill the existing technology gaps, methods to predict the behavior of drilled and grouted piles will include a great deal of engineering judgement and site specific experience.

Several considerations are of concern, including:

1) Most of the available analytical and design methods are based on a high degree of empiricism, intuition, and practical experience.

2) Publicly available information on offshore load tests on long drilled and grouted piles is insufficient.

3) Most data used in understanding drilled and grouted pile behavior are based on short length bored piles, drilled shafts and soil anchors.

4) Quantitative assessment of load carrying capacity degradation for drilled and grouted piles under cyclic tension is not publicly available.

Several technology gaps need further research and development, they include:

(1) Understanding effects of installation (drilling, grouting, regrouting, etc.) on the state of stress and strength properties of the surrounding soils.

(2) Understanding the interface behavior of steel-grout and grout-soil under tension and under different surface conditions.

3) Understanding the failure mechanisms of drilled and grouted piles under different soil and installation conditions.

Parallel efforts involving the backfit between analyses and experiments will be necessary to provide answers to the above questions.

Conclusions

Notwithstanding the above concerns related to technology gaps and the relative paucity of data on long drilled and grouted piles, these foundation members appear to be a viable and attractive solution to anchor TLPS (and other types of structures) in deep waters. Large scale field tests, such as those sponsored by Conoco, in which actual drilling, grouting and offshore systems and operations can be checked, will be needed to improve the level of confidence in the design of drilled and grouted piles for offshore applications. It is hoped that information on presently proprietary tests will be released for publication within the not to distant future so that the level of reliable data on offshore drilled and grouted piles can slowly raise toward the level attained by driven piles.

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1.0 INTRODUCTION

1.1 General

As part of their effort to review presently available Tension Leg Platform (TLP) technology, the Minerals Management Service (MMS), through Sandia National Laboratories (SNL), requested that the Earth Technology Corporation (ETC) assess the state-of-the-art on drilled and grouted piles for tension leg platform (TLP) foundations. The Earth Technology Corporation was previously contracted by SNL to review the state-of-the-art in foundation design and analysis for TLPs and vertically moored platforms (Earth Technology, 1983a). The present study is an extension of the latter, focusing on the state-of-the-art on drilled and grouted piles.

1.2 Background

With the increasing worldwide demand for oil and gas, offshore drilling and production facilities have been moving farther offshore into deep waters. Conventional fixed jacket platforms become less economical in deeper water. Compliant structures such as TLPs become economically and technically more attractive with increasing water depth.

The first TLP (Fig. 1-1), installed by Conoco in the Hutton Field of the North Sea (Hart el al., 1985), consists of a buoyant drilling platform anchored to seafloor templates by mooring tendons which are pretensioned to minimize platform motions. The seafloor templates are secured by driven piles. Drilled and grouted piles, gravity-type foundations or a combination of both along with driven piles are also being considered to anchor future TLPs.

Driven piles have been commonly used to anchor offshore structures. Offshore experience on installation techniques and behavior under different loading conditions is thus greater for driven piles than for any other type of foundation. However, with an increase in water depth, pile driving becomes a problem due to the unavailability of a proven underwater hammer for deep water. Drilled and grouted piles are, therefore, being considered by the industry to anchor TLPs at deep water sites. Since installation and grouting of surface and well casings for deep water oil and gas exploration has proven routine, the same technology could successfully be adapted for installation of drilled and grouted piles.

For a TLP foundation, the load imposed on piles consists of a static tension bias with an additional cyclic component. The pile behavior under such a load is not well-understood, especially for drilled and grouted piles. The design of drilled and grouted piles is based on empirical correlations from a limited number of onshore load tests on piles which were much shorter than those planned for future TLP foundations. Consequently, the validity of the data base for application to TLPs is questionable.

1.3 Scope and Objectives of Study

The objective of this study was to document the state-of-the-art on installation techniques, load-deformation behavior, and quality control of drilled and grouted piles for deep water TLP foundations. The approach consisted of a survey of recent literature and a review of the construction practices in the industry. The industry's experience on installation of offshore oil well casing is included for its relevance to installing surface casing for drilled and grouted piles.

1.4 Contents of Report

This report contains the results of our state-of-the-art study on drilled and grouted piles for anchoring tension leg platform foundations. The information in this report is presented in the following chapters:

1.0 Introduction
2.0 Overview of Industry Experience
3.0 Drilling Technology
4.0 Grouting Technology
5.0 Analysis and Design Methods
6.0 Major Technology Gaps and Recommended Research and Development

Chapter 1.0 (this chapter) presents the project background, scope and objective of the study, contents of this report, and the project team who participated in the work. Chapter 2.0 discusses particular considerations associated with the design of a TLP, such as loading conditions, template and foundation piling configuration, performance requirements, soil conditions and water depths. An overview of the construction and petroleum industry's experience in designing and installing drilled and grouted piles and grouted casings is also given. Chapters 3.0 and 4.0 present the present state-of-the-art in drilling and grouting technology, respectively, with particular attention to installing drilled and grouted piles. Chapter 5.0 discusses the analytical and design methods presently used to design drilled and grouted piles. This includes a discussion of pile-grout-soil failure mechanisms, and axial and lateral behavior of drilled and grouted piles. Chapter 6.0 presents a discussion of the major technology gaps facing the drilling, grouting, design and quality control aspects of installing drilled and grouted piles and proposed research to answer the technology gaps.

1.5 Project Team

The project team responsible for performing this study included several personnel from The Earth Technology Corporation (ETC). Dr. Jean M.E. Audibert of ETC, Project Manager for the study, directed the technical effort, developed the first draft of the report and provided overall review of the various manuscripts. As co-investigator, Scott Bamford produced the second draft with updated information on industry experience and the state-of-the-art in designing and constructing drilled and grouted piles. Hudson Matlock and Dr. Bill Lu provided peer review of the second draft. The final text was summarized by Dr. Chairat Suddhiprakarn based on the information from the second draft with some additional material. Scott Bamford and Dr. Audibert provided a review of the final text.

1.6 Acknowledgement

This report was compiled with the input from numerous individuals from the oil and gas industry and its service companies. The contributions of these individuals and companies and their willingness to discuss the industry's outlook with regard to TLPs, offshore construction, and drilled and grouted piles is gratefully acknowledged.

2.0 OVERVIEW OF INDUSTRY EXPERIENCE

2.1 General

This chapter summarizes the background information and industry's experience in the use of drilled and grouted piles to support offshore structures. The primary sources of information are from published materials, experience of the project team, and industry interviews.

2.2 Background

Background information relating to the design and installation of drilled and grouted piles for TLP foundations are listed below:

2.2.1 TLP Loading Conditions

Various factors affecting the design of a TLP foundation are as follows:

Platform Configuration. The size of the floating structure determines the surface area exposed to waves and winds, and hence, vertical loads induced in the tendons. Similarly, the size of the foundation template affects the lateral loads imposed by bottom currents and possible mudflows in geotechnically unstable areas.

Payload. The additional weight carried by the floating structure will dictate the buoyancy force to be carried by the anchoring systems.

Environmental Considerations. Perhaps this is the most important factor. The loads derived from wind, waves and currents can vary significantly depending on the particular offshore location. The orientation of the structure relative to the design storm direction will also dictate the magnitude of the design load.

<u>Water Depth and Horizontal Offset Restriction</u>. In deeper water locations, the overall stiffness of the riser and tendon members decreases as compared to shallow water sites, thus, allowing more horizontal movement of the floating structure.

<u>Soil Conditions</u>. Site soil conditions will influence the stiffness required for the foundation templates. It will also dictate the number, diameter, and length of piles used.

The design and installation of drilled and grouted pile foundations will have to include consideration of the unique loading conditions of a static tension bias with added cyclic component under both operating conditions (long-term cyclic) and maximum storm wave loading (short-term cyclic).

2.2.2 Template Configurations and Performance Requirements

Template configurations presently being considered to anchor TLPs include the following:

1) Single piece template combining the well area and tendon anchor foundations.

2) Two-piece template consisting of a well template and one template frame combining the tendon anchor foundation.

3) Three-piece template consisting of a well template and two separate templates, each composed of two tendon anchor foundations.

4) Five-piece template consisting of a well template and four independent tendon anchor foundations.

Multiple template configurations allow the use of lighter structures, thus requiring smaller and less costly floating support in the installation process. Single template configurations, on the other hand, need larger floating support but do not have template-to-template misalignment problems which can affect the load distribution among the tendons.

2.2.3 Pile Geometry and Installation Requirements

The design of TLP pile foundations to resist axial and lateral loads is defined by the pile diameter, wall thickness and penetration. In a normally consolidated clay where the soil strength increases with depth, long, small-diameter piles may be the most effective due to a rapid gain in capacity with depth. For piles dominated by lateral loading, shorter, large-diameter piles may be more suitable because most of the lateral load transfer is concentrated near the seafloor. In either case, drilled and grouted piles could be a favorite choice because of the existing equipment and techniques available to meet virtually all penetration and water depth criteria.

Recent in-house research by The Earth Technology Corporation has shown that drilled and grouted piles with 48 to 54-in. diameters and 250 to 450 ft embedment are necessary to anchor a TLP off the U.S. West Coast or in the Gulf of Mexico. Such sizes are within the feasible range of present drilling practices. The required pile penetration for a typical well casing (20-in. insert pile in a 26-in. hole) could be in the range of 600 ft in the Gulf of Mexico if conventional well casing sizes were used.

Drilled and grouted piles can be installed to within ± 1 degree of vertical. This error, although having little effect on pile capacity, may cause problems in a closely spaced group of long piles when one pile may run into another during drilling.

2.2.4 Soil Conditions

The soil conditions present at potential TLP sites around the world vary from soft clays to gravelly sands. These areas include the following:

1) Normally consolidated, highly plastic clay (very soft to hard) as found in areas of the Gulf of Mexico. Shear strength increases linearly with depth.

2) Overconsolidated plastic clay as found offshore southern California, in the North Sea, and in the Gulf of Mexico. Shear strengths may reach 2 ksf at the mudline, as in the North Sea.

3) Non-plastic to low plasticity silts similar to those offshore southern California such as in the Santa Maria Basin. Soil strength parameters, such as the effective friction angle ranges from 30 to 35 degrees.

4) Cohesionless soils from sand to gravel as in some areas offshore Alaska and in the North Sea. Effective friction angles range from 35 to 40 degrees.

5) Special soils such as silts, volcanic ashes, and calcareous sands which may be encountered in areas such as offshore Alaska, Japan, and the tropics, respectively.

2.3 Industry Experience

To date, driven piles have been used for anchoring offshore facilities more often than drilled and grouted piles because of the successful experience, continued improvement in installation equipment and techniques, and the practical concerns associated with drilled and grouted piles. Nevertheless, drilled and grouted piles have previously been used as foundation members for offshore structures (Young et al, 1978; Duvivier and Henstock, 1979) and to temporarily anchor drilling vessels. The principal geographical areas of application and the soil conditions encountered were as follows:

Geographical Area	Soil Conditions
North Sea	Stiff to hard clay, dense to very dense gravelly sand and sand with gravel layers
Mediterranean Sea	Interbedded sand, silt and normally consolidated clay

Offshore Australia	Calcareous soils
Arabian Gulf	Interbedded hard clay, gypsum, dense to very dense sand, and calcareous soils
Offshore California	Dense sand and silt with layers of stiff to hard clayey silt
Offshore Brazil	Calcareous soils

Drilled and grouted piles were the foundation of choice at the above locations because of the hard driving conditions present and the anticipated loss of pile capacity (such as in calcareous soils) if driven piles were to be used.

Drilled and grouted piles have also been used as permanent mooring anchors for offshore drilling rigs and vessels because of readily available onboard equipment (McLamore et al., 1982). However, these piles are typically short and are installed in shallow water zones.

Bored piles, drilled shafts and anchor piles have been installed onshore and in shallow water on a routine basis for transportation, commercial and other applications. The available information, although useful, would require additional correlation for use in the TLP design due to the differences in pile size and in the nature of the applied loads.

2.3.1 Offshore Drilled and Grouted Piles

The design of a drilled and grouted pile for offshore use may be as follows:

Insert piles. A borehole is drilled to the penetration depth, an insert pile is lowered into the hole and the annulus between the pile and the soil is grouted from the bottom up. Alternately, the pile tip can be fitted with an expendable cutting tool, and the oversized hole drilled simultaneously with the insert pile installation. This avoids drilling-related problems such as positioning and re-entry and saves the time required to trip the drilling equipment and to run the insert pile.

Composite drilled and grouted piles. These piles are similar to insert piles except that an open-ended surface casing is first driven or jetted to a specified depth (or until refusal) and forms the upper section of the foundation pile in soft or unstable soils. An oversized hole of smaller diameter than the surface casing inside diameter is then drilled to the final penetration depth. An insert pile is then inserted into the borehole and grout is placed in the annulus between the insert pile and formation wall, and between the surface casing and insert pile.

Belled piles. This is a modification of an insert pile or a composite drilled and grouted pile to increase end bearing and uplift capacity. A bell may be constructed at the tip of a pile by underreaming or flaring of the formation with an expander tool and then filled with grout. The upper section of the pile is usually formed by a driven surface casing.

2.3.2 Offshore Oil Well Installations

Drilled and grouted piles have generally been installed offshore to water depths of less than 500 ft. Oil well casings for exploration have been installed in water depths to 7,500 ft (Collipp et al., 1984). In both cases, a surface casing is used to stabilize the borehole in the upper, weak sediments. For exploratory wells, the surface casing is usually 30-in. diameter and is typically driven in place (for shallow applications), jetted in place, or drilled in and then grouted to the formation. Penetration of the surface casing typically ranges from 50 to 400 ft depending upon the requirement for a hydraulic seal.

Surface casing installation using external jetting can cause severe erosion and cratering (forming of bellmouth holes) at the seafloor and can lead to unacceptable lateral deflection or settling of a drilling template for production wells. Some casings are, therefore, installed using internal jetting.

On the other hand, the present limit of operation for underwater hammers is around 1,500 ft. Although studies are being carried out to extend the operating range to perhaps 2,000 ft, other methods are still necessary for the installation of the surface casing in deep waters (2,000 ft plus).

2.3.3 Onshore and Coastal Experience with Drilled and Grouted Piles

Bored piles (bored and cast-in-situ), drilled shafts, piers, and micropiles (soil anchors) have been extensively used onshore and for waterfront structures. The experience gained provides an important data base in understanding the effects of installation techniques on drilled and grouted pile behavior.

Offshore drilled and grouted piles are similar to conventional soil anchors where anchor rods are grouted into pre-drilled holes (Hanna, 1982). A significant amount of data indicates the advantages of using high pressure grouting to increase the holding capacity of soil anchors. Researchers have had some success with high pressure grouting for increasing pile capacity by a factor of 2.5 or more (Gouvenot and Gabaix, 1975). Expansive grouts have also been used successfully in tighter formations without high pressure grouting to increase axial capacity by as much as 25 to 75 percent.

2.3.4 Applicability of Past Experience to TLP Piles

There is limited information regarding design, installation and performance of drilled and grouted piles in deep water. Figure 2-1 summarizes the installation times and problems encountered during installation of drilled and grouted piles in the North Sea at the Thistle A and Piper platform sites with water depths of 530 and 474 ft, respectively (Richardson, 1977; Santa Fe International, 1977; Young et al., 1978; Richardson et al., 1978; Edwards, 1978; Duvivier et al., 1979; St. John, 1980). Some of the problems encountered could be avoided in future projects by using seafloor (rather than deck) discharge of drilling fluid and grout.

Notwithstanding the above experience with installation problems, drilled and grouted piles are believed to have some definite advantages over driven piles for TLP applications. Existing equipment and techniques in oil well construction could be utilized with a minimum development effort in contrast to a substantial effort which would be required to develop underwater pile driving equipment for deep water. Also, if the installation is such that the TLP tendons need to be hooked up immediately after pile installation, drilled and grouted piles would have immediate capacity higher than that of driven piles which require significant time for soil setup before reaching their ultimate capacity. The latter is due to the effect of cavity expansion and soil reconsolidation associated with the process of pile driving. Installation of a drilled and grouted pile, if performed properly, does not significantly change the state of stress in the soil region adjacent to the pile. Therefore, there are few time dependent effects associated with drilled and grouted piles.

3.0 DRILLING TECHNOLOGY

3.1 General

This chapter gives a summary of the industry's experience in drilling technology, most of which is related to oil well technology. This information, especially that presented on drilling procedures, potential problems and technology gaps, is felt to be valuable for TLP application, as previously discussed.

3.2 Drilling Equipment and Procedures

The offshore oil and gas industry's capability in deep water exploratory drilling is continuously advancing. Exploration wells for Amoco and Shell were drilled to a water depth of 7,500 ft in Wilmington Canyon and Baltimore Rise lease area off the U.S. east coast (Petroleum Engineer International, 1986). During the Deep Sea Drilling Project, drilling was performed to obtain soil and rock samples to 2,850 ft below seafloor in water depths up to 18,000 ft on the Bermuda Rise in the western North Atlantic (Rabinowitz et al., 1986).

It is expected that a semi-submersible drilling rig or drillship would be used to install the foundation templates and piling for a TLP in the water depths under consideration. In 1986, the following number of vessels were available to drill in the cited water depths (Ocean Industry, 1986):

	Vessel Types	
Water Depth	Operating (under construction)	
ft	Drillship	Semi-submersible
1,000 to 2,500	15	102(6)
2,500 to 4,000	8	5(4)
4,000 to 7,500	3	4(2)
7,500 to 10,000	-	1

The above drilling vessels are typically equipped with a reverse-circulation drilling fluid system and equipment rated to 20,000 ft drilling depth. Drill rig drawworks are rated between 4,000 to 6,000 installed horsepower. The range of equipment would be more than adequate for drilling 26-to 54-in. diameter holes to 300 to 600 ft penetration depth for installing drilled and grouted piles.

Reverse circulation drilling process, which pumps the drilling fluids down a borehole annulus with returns up the drill string, was used to drill the piles at the Thistle A and Piper platforms (Figure 2-1). Problems associated with this type of drilling procedure were as follow:

1) Loss of circulation and mud during drilling due to overpressure,

2) Difficulty in returning drilling mud to the deck level, which may end up in a complete abandonment of the use of drilling mud.

3) Cave-ins due to insufficient unit weight or viscosity of the drilling mud.

The piles on the Thistle A and Piper platforms were drilled through the platform jacket legs which served as the outer casing for the drilling fluid. In deep water, it would not be practical nor advisable to return the drilling fluid to the drill floor.

Likely, the borehole annulus will have to be left open to the sea water hydrostatic head in order to limit the downhole pressure. Suitable methods for removing soil cuttings accumulated around the template will need to be devised. Such cuttings could interfere with the subsequent drilling of nearby wells or the operation of equipment located on the template.

3.3 Surface Casing Installation

Currently, several methods are commonly used for installing surface casings from floating drilling rigs. Other methods are available but may require more developmental work or may have some practical limitations.

3.3.1 Fixed Jetting Assembly

This is the most commonly used technique (Fig. 3-1). It consists of a fixed jetting head attached to the tip of the casing to be installed. The jetting head may be either a non-retrievable type (which can be drilled out later) made of soft aluminum, or a retrievable steel unit.

The drill string and casing assembly is lowered to the seabed. The mud pumps are started, using the casing as a conduit to the jetting head. The whole assembly is jetted into the seafloor with the mud returning up the annulus between the borehole wall and casing.

This is the least costly method due to its simplicity and speed of operation, particularly in soft surficial sediments such as found in the Gulf of Mexico. It is a proven technique familiar to most operators and drilling crews. There is little danger of the casing becoming stuck since the borehole will typically be much larger than the casing due to the high-volume and high-pressure jetting process.

The disadvantages of the method include the potential to form a crater at the surface. A crater of 5 to 10 ft diameter is common and 50-ft craters have been reported. The technique has no control over verticality of the borehole. It also produces an irregular and frequently oversized hole, which will result in a lower quality foundation.

3.3.2 Drilled-In Conductor

As shown in Figure 3-2, the borehole is first drilled, followed by the casing assembly grouted in place. Sometimes a smaller diameter pilot hole is first drilled as a guide. This is a two trip operation (three if a pilot hole is used), one to drill the hole and one to run the casing. Between trips, the hole is open and exposed to sloughing of the surficial sediments.

Advantages of this method include the capability to control verticality of the borehole due to the pendulum effect of drill collars, especially when using the pilot hole option. The method produces the most uniform borehole diameter with only small cratering effects.

Among the disadvantages of the method are the longer time required because of additional tool trips in the process. Hole filling and sloughing, particularly in soft unconsolidated soils, is also a problem. Heavy drilling fluid can be used to control sloughing, but at a higher cost.

3.3.3 Drilling with a Turbo-Drill

This is a one trip process similar to conventional jetting except that, in place of a jet head, a turbo-drill (or casing drill) is attached to an expanding, under-reaming bit (Fig. 3-3). The bit is positioned just slightly ahead of the casing. The returns can either be taken internally up through the casing or externally up the annulus between the casing and the borehole wall. The method produces a uniform size borehole with less likelihood of sloughing or pipe sticking (as in the drilled-in method). The operating time is also shorter.

The turbo-drill, however, is fairly expensive to operate even on a rental basis. A large capacity mud pump with at least 3,000 total horsepower is typically required to power the unit. The method offers only moderate directional control and also causes significant crater formations if returns are taken externally. With internal returns of soil cutting where the hole is slightly larger than the OD of the casing, skin friction from the soil may limit penetration prematurely.

3.3.4 Jetting with Internal Returns

This is a modification of the conventional jetting method (Fig. 3-4). The jet head is retracted inside the casing and returns are taken internally inside the casing.

This system produces a narrow borehole only slightly larger than the casing outside diameter. No special equipment is required, and the operation is fairly quick as with the turbo-drill. Complete elimination of crater formation has been reported with the use of this method.

The method has some disadvantages, however. There is no reliable degree of directional control. The positioning of the jet head is critical to the success of this technique. Tight tolerance between the casing OD and borehole wall increases the probability of pipe sticking. Drilling crews are generally not familiar with the technique and, hence, some training is necessary. However, this technique has been used in the Gulf of Mexico by a limited number of operators without any reported problems.

3.3.5 Expendable Casing Drill

An expendable drill bit is attached to the tip of the casing. Advancing the casing assembly can be done in a single drilling operation, with the soil cuttings removed through the inside of the casing.

This method has been used for installation of small piles to shallow penetrations (less than 100 ft) on land (Tomlinson, 1977) and offshore (McLamore et al., 1982). The advantage of this method is that significant penetrations are possible in all soil types since the hole is drilled slightly larger than the casing diameter, thus eliminating the soil resistance on the casing wall. This method has had only limited field testing to date and may require further development to control the size of the annulus between casing and soil, thus optimizing the skin friction.

3.3.6 Suction Method

The suction method is based on the application of a reduction in the pressure inside the pile, similar to the scheme proposed by Senpere and Auvergne (1982). This method has the potential for installing surface casings to 100 ft or more penetration in normally consolidated clay. The force used in the installation results from the ambient seawater pressure which is considerable in deep waters. Suitable deep water pumps would have to be developed to evacuate the water from within the casing.

This method has been used for only limited penetrations, typically less than 50 ft, and will require extensive further studies.

3.3.7 Vibratory Technique

Vibratory hammers have also been used in some occasions particularly in shallow waters to drive piles and to pull out stuck casing. Hydraulically driven vibratory hammers, which can be tuned to resonance and can impart to the casing a wide range of input motions from single pulses to random signals, may have the greatest potential for deep water applications. Substantial development work on underwater vibratory hammers will be needed, however, in order to apply the technique in deep water.

3.4 Spatial Tolerances

Borehole misalignment is usually expressed in terms of deviation and dog-leg. Deviation is defined as the total off-vertical angle, while dog-leg is the deviation over any 100 ft segment of the well and includes both inclination and azimuth. Dog-leg usually occurs in stratified soft and strong soil and rock deposits or in soft deposits when boulders or trees are encountered.

Typically, holes in which casing strings are run have less than one degree of deviation. A similarly strict to perhaps more strict requirement will be necessary when constructing a pile group for a TLP. For a 600 ft long pile, the pile tip could deviate about 10 ft horizontally under the above criteria. This deviation may not be acceptable depending on pile group configuration. Therefore, methods for better alignment control should be further evaluated.

Drilled hole diameters are generally on the order of 2 to 4 in. greater than the bit size in the more stable soils. This oversize can be much greater in soft sediments (as found in the Gulf of Mexico) due to wash-outs and bit wandering. On the other hand, squeezing of the hole can also occur if mud weight is not sufficient, as further discussed later.

The selection of the drilling assembly also affects verticality of the borehole. To meet the verticality requirements for a larger borehole (usually greater than 50 in., depending on soil conditions), a multiple trip approach may be necessary starting with a smaller pilot hole (Section 3.3.2 and Fig. 3-2). A surface casing, which is commonly used to stabilize the borehole in soft surficial soils, is also helpful in controlling hole verticality.

Another viable installation scheme, which combines the insert pile, drill bit, and grout facilities in the drill-stem setup, has been used in small anchor piles to moor drilling vessels (McLamore et al., 1982). In this scheme, the process of drilling, pile insertion, and grouting can be performed without interruption by tool removal or makeup. In addition, the drill bit can be withdrawn through the inside of the insert pile at the completion of drilling. Such an approach would minimize the time the borehole has to stay open, thus reducing the risk of borehole cave-in.

3.5 Quality Control

Quality control of the drilling operation will be extremely important to the successful performance of a drilled and grouted pile. Designs using direct TLP tendon to pile connections will require 100 percent installation success. Good installation procedures will have to be developed and fully tested in deep water to establish confidence. The collection of information from exploratory well installations in all water depths should be a high priority item.

Verticality of the borehole is another important factor especially in a tight pile group. While it seems feasible to drill a vertical hole to within ±1 degree, it may be desirable to instrument the drill string with inclinometers to monitor verticality during installation and further expand the data base.

Boulders and tree trunks encountered during drilling in the North Sea and in the Gulf of Mexico have been a significant problem. Existing geophysical methods may need to be modified or new ones developed to better detect such obstructions.

3.6 Technology Gaps in Drilling

The North Sea experience at the Thistle A and the Piper jackets (Fig. 2-1) and information gained from drilling offshore oil wells was utilized to identify the technology gaps present in the drilling industry today for installing drilled and grouted piles. These gaps are discussed below.

Surface Casing Installation. As previously discussed, a surface casing is necessary in weak sediments such as normally-consolidated clays. Different installation methods were described in Section 3.3. Considering the various factors such as underwater hammer problems, crater formation from external jetting processes, etc., the viable approaches for installing surface casings in weak sediments and cohesionless soils could be one or a combination of the following:

1) Drilling a large borehole through the template sleeve, then running the surface casing. This method is appropriate only if the borehole can remain stable for a short period of time as may be possible in stiff surficial or overconsolidated clays.

2) Using the casing drill method as described in Section 3.3.3 with internal returns of soil cuttings to minimize crater formations.

3) Using an expendable drill bit (Section 3.3.5).

4) Using the suction method (Section 3.3.6).

Five areas of development are required to improve the understanding of design considerations for surface casing for TLP piles. They include:

1) Improve understanding of borehole stability with time.

2) Improve understanding of the relationship among soil erosion, soil type and casing installation method.

3) Develop a reliable method to determine required casing penetration.

4) Modify existing casing installation methods or develop new techniques.

5) Optimize the design details of the template-casing-insert pile connection.

All these development needs are interdependent and should be assessed simultaneously.

<u>Seafloor Cratering</u>. During jetting of surface casings, cratering usually occurs because of high flow rates (as high as 900 gpm) and high jetting pressures used to advance the casing. The latter causes displacement of large amounts of seafloor material from around the casing. A drilling mud program in which the drilling mud and soil cuttings are discharged onto the seafloor can also lead to sloughing of the borehole at the surface where craters may form. In the Gulf of Mexico, it has been reported that a 50 ft diameter crater is possible when attempting to jet in 30 in. diameter casings.

Seafloor cratering reduces the amount of lateral and axial support available to the casing itself and is also detrimental to template leveling and mudmat support by the seafloor (Fig. 3-5). Thus, modifications to casing installation methods is likely necessary to assure proper casing installation.

<u>Borehole Stability</u>. Borehole instability is caused by the imbalance of stresses along the borehole wall due to the relief of lateral pressure, and subsequent pore water migration which causes a reduction in soil strength. If the soil shear strength is not sufficient to withstand this stress imbalance, the hole collapses. Casing or drilling mud is required to resist this stress imbalance and stabilize the borehole before the pile is inserted and grout injected to cement the casing in place.

Drilling mud can also help inhibit the migration of water towards the borehole wall and minimize swelling. A light-weight mud, consisting of seawater with only a small amount of gel and/or weight material, is generally used while drilling through the surface sediments. Because the unit weight of these soils is typically in the range of 80 to 100 pcf (10 to 13 ppg), the drilling mud is usually kept at the unit weight of 9.5 to 12 ppg to provide sufficient lateral pressure. Experience from geotechnical site investigations in the Gulf of Mexico has shown that mud weights in the range of 9.5 to 10.5 ppg are generally sufficient to produce a good hole for foundation investigations.

Although stability of the deeper layers of soil is somewhat better because the soil formation has been subjected to higher all around confining pressures for longer periods of time, sloughing may still occur if sufficient mud weight is not maintained. This event may lead to squeezing or enlargement at various depths along the borehole causing a nonuniform grout annulus, and finally a poorly constructed pile.

Borehole Induced Stress Relief. Drilling a borehole causes lateral stress relief and expansion of the soil adjacent to the borehole wall leading to a borehole caving in. This can happen in all soils, especially in cohesionless soils. Stress relief can also soften the soil near the borehole due to moisture increase from the drilling and grouting operations. Whereas the latter problem is normally encountered in overconsolidated clays, it is also possible at depth in normally consolidated clay where very stiff, relatively low moisture content soil is present. These problems result in a reduction of load transfer between pile and soil.

Drilling Mud. The use of drilling mud must be carefully controlled to limit the following unfavorable effects:

1) Hydraulic fracturing of the surrounding soils.

2) Formation of a mudcake at the borehole wall which could negatively affect the load carrying capacity of the pile.

3) Migration of drilling mud and water into the borehole wall, which may cause a reduction in soil strength and thus load transfer capacity.

4) Squeezing of the borehole toward the insert pile due to insufficient mudweight causing channelling and loss of grout contact with pile wall.

5) Insufficient gel strength or viscosity to lift all the cuttings, thus causing possible blockage and channelling of grout.

The most serious concerns above are hydraulic fracturing and mudcake removal, which are discussed in the following sections.

Hydraulic Fracture. Hydraulic fracture occurs when the lateral pressure of the drilling fluid exceeds the resistance of the soil formation, causing local fracture and intrusion of these drilling fluid along the borehole wall. This was one of the major problems encountered during installation of drilled and grouted piles in the North Sea. The consequences of hydraulic fracture include:

1) Excessive mud loss.

2) Mud contamination of the adjacent soils.

3) Oversized hole.

4) Instability of borehole.

Efforts have been made to estimate pressures at which fracture occurs. The industry's understanding of hydraulic fracture in soils, especially in silt and sands, is limited. Currently, the following equations are used to estimate vertical and horizontal fracture pressures (Matthews et al., 1967):

Cohesive Soil

o $P_{hf} = K_o \gamma' Z + 2S_u$ (vertical fracture) (3-1)

o $P_{hf} = \gamma' Z + 2S_u$ (horizontal fracture) (3-2)

Cohesionless Soils

o $P_{hf} = K_o \gamma' Z$ (vertical fracture) (3-3)

o $P_{hf} = \gamma' Z$ (horizontal fracture) (3-4) where:

P_{hf} = effective fracture pressure due to column of drilling mud
K_o = coefficient of earth pressure at rest
γ' = effective unit weight of soil
Z = depth below seafloor
S_u = undrained shear strength of soil

The above equations are too simple to fully represent the complex behavior of hydraulic fracture in soil but are sufficient to aid in design of drilling and grouting operations. To estimate reliable hydraulic fracture pressures, further work is required to study the relationship between fracture pressure and strength-deformation properties of the soil, as well as to understand fracture propagation patterns and flow characteristics.

To alleviate the hydraulic fracture problems, light weight mud should be used, if possible, to reduce mud pressures. Special flow pipes and valves might be designed to decrease pressure gradients. In addition, return of soil cuttings directly to the seafloor rather than to the drill floor (as in a reverse circulation process used at the Thistle A and the Piper platforms) is recommended. However, care should be taken to avoid soil cuttings covering the template, as well as mud flow problems in the adjacent boreholes. Thus, development of a workable cuttings removal system may be necessary. Use of economical and ecologically acceptable muds would also be desirable since large quantities will be disposed at the seafloor.

<u>Mudcake and Drilling Mud Migration</u>. The formation of mudcakes or filtercakes on the borehole wall during drilling operations has generally been regarded as advantageous to prevent loss of circulation. However, available field and laboratory test results (Fig. 3-6) have indicated that a mudcake may greatly reduce the shearing resistance at the soil-grout interface of a drilled and grouted pile (Tucker and Reese, 1984). Similar effects in cohesionless soils were also reported by Fleming and Sliwinski (1974). Several studies (Reese et al., 1976; Sliwinski, 1977; Touma and Reese, 1974) reported no significant differences in shaft load transfer when bored piles were drilled with bentonite mud or with the hole left open and dry (onshore application) in such soils as Beaumont clay, glacial till and sand. Therefore, it is

difficult to assess whether or not mudcake formation or soil softening due to an increase in moisture content reduces shearing resistance.

Various studies have shown that the grouting technique could influence the effect the drilling mud has on the load capacity of a drilled and grouted pile. Mud film can be removed by scouring and sweeping of the borehole wall during grouting. Flow velocity, viscosity of the drilling mud, as well as the pump pressure would affect the extent of mud cake removal.

For TLP applications, flowrates and pump pressures of drilling mud and grout may not be high enough to remove any mudcake built-up on the borehole wall. Multiple stages of high-pressure grouting could possibly displace and/or consolidate the mudcake. In addition, seawater with gel could be used as the drilling fluid to prevent mudcake formation.

<u>Hole Verticality</u>. Problems of hole verticality should be addressed to eliminate (1) interference problems between boreholes, (2) difficulties in centralizing the insert pile, and (3) the potential for causing excessive bending stresses in the pile. Although using proper drilling assemblies and surface casing can alleviate the problem, further research and development is required.

<u>Mechanical Disturbance</u>. Disturbance of the soil during drilling, tripping of drilling tools, and running casing could reduce the load transfer capacity of drilled and grouted piles. Although quantitative study on these effects is not feasible, the degree of disturbance could be minimized by (1) using proper drilling pressure for hole stability, (2) using an optimum drilling rate slow enough to prevent partial collapse and wash-out of soil, and fast enough to shorten the time during which the hole stays open, (3) slowly withdrawing the drilling bit to allow the drilling mud to flow around the drill bit, and (4) using stabilizers or centralizers along the drill string to minimize "whipping" of the drill string.

An installation scheme which combines the insert pile, drill bit, and grouting facilities in the drill-stem setup (Section 3.4) could minimize mechanical disturbance.

Presence of Boulders. This problem is common in the northern and central area of the North Sea, and occasionally is present in other areas such as Alaska. It typically results in drilling delays, dog-leg problems and frequent hole abandonments. For TLP pile installations, this could be a serious problem because it is almost impossible to relocate the hole once piling operations have begun unless extra slots are provided in the template to allow installation of replacement piles. In the latter case, the template structure should be so designed as to allow for such changes in load distribution.

Sub-seafloor seismic surveys can usually detect large near surface boulder accumulations (about 10 to 20 ft below the mudline), but not isolated boulders, however. Short of abandoning the hole, the method commonly used to overcome the boulder problem is to slow the drilling rate, and use light bit weight, high rotary speed, and heavy bottom hole assemblies to achieve maximum pendulum effect.

Surface Crusts and Near-Surface Lenses. Hard thin surface crusts and near-surface lenses (prevalent in the Persian Gulf) are troublesome especially in shallow water because there is typically not enough vertical distance to achieve the necessary bit weight. For TLP locations, the large water depth allows enough bit weight to be developed and the problem will reduce to choosing a compromise bit design to minimize hole diameter fluctuations, which usually result from abrupt changes in formation density and penetration rate.

Highly Fractured Formations. These are typically shale formations which frequently occur in seismically active areas such as the west coasts of North and South America, and in coal seams of the North Sea. In deep water drilling with closed circulation techniques, fractured formations can lead to serious loss of circulation. In open hole surface drilling, which would likely be used in TLP piles, irregular hole diameter could result due to the broken fragments of the formation falling into the hole, making the hole difficult to clean out. It may be necessary to start with an undersized hole in a multiple trip process. Fissures may also open as a result of drilling and could lead to excessive mud and grout loss and a poor grouting job.

Swelling Shale and Clay. Swelling results from the chemical reaction of clay minerals with water and frequently occurs at shallow depths. This could lead to excessive time spent in reaming the hole and, at worse, loss of the hole. In TLP pile installations, where the hole will be drilled open to the seafloor, a common method of using oil-based or synthetic muds would not only be costly, but could also pose environmental problems. Usually, the exploratory wells drilled prior to the TLP pile installation will give advance warning if swelling shales or clays are present. The pile installation operation can then be planned accordingly.

Dipping Formations. Dipping formations could cause a deviation from vertical while drilling. This could be controlled by using a suitable configuration of the drilling assembly. The most difficult situation would be a sharply dipping formation in the first 300 ft of the hole where a wash-out near the surface would prevent using stabilizers to configure pendulum or packed-hole drilling assemblies. The best procedure in such a case is to rely on the records from previous drilling in the area, if available.

Salt Formations. Encountering salt deposits while drilling an open hole with sea water usually results in dissolution of salt deposits present in the formation causing a cavity or cavern in the borehole wall. A common solution to the problem is to drill the hole with a saturated brine. This would be difficult and expensive in an open hole where drilling fluid is vented to the seafloor. This potential problem should be studied carefully, with assistance of experienced mud engineers.

Hydrates. Under the combination of temperature and pressure existing near the seafloor at deep water locations, the existence of hydrates in the surficial sediments is likely. Hydrates have been encountered at several deep water sites in the Gulf of Mexico. The hydrates exist in a metastable condition; any changes in the temperature or pressure during drilling may result in the sublimation of the hydrates in the formation, resulting in borehole instability. At present, little is know of the effects of drilling on the formation, although severe disturbance has been noted in core samples. For such sites, additional studies are recommended.

4.0 GROUTING TECHNOLOGY

4.1 General

Available grouting techniques for cementing surface casings and drilled and grouted piles vary significantly. The grout formulation and procedures are usually tailored to meet specific job requirements.

For offshore drilled and grouted piles, grouting of the pile-borehole annulus is similar to the grouting of an oil well casing. The initial or primary grouting is done by gravity-assisted pumping, sometimes in stages to avoid hydraulic fracture of the soil. Secondary or high pressure grouting can then be performed as a remedial measure to correct areas along the pile which require improvement. High pressure grouting has proven to increase the frictional capacity of piles and is worth further consideration.

4.2 Grout Placement Procedures

4.2.1 Inner String Method

The inner string method (Fig. 4-1) uses a single retrievable grout line assembled inside the pile or casing with a sealing adaptor (or float shoe) at the bottom of the pile. Ball or plug-operated diverter valves are placed on the grout line for use in multiple stage grouting.

Inner string grouting allows efficient, bottom-up filling of the pile borehole annulus in one stage or in several stages as required to prevent hydraulic fracturing of the soil. Finally, the entire length of grout line can be disconnected from its seals and used to fill the inside of the pile, if required. The entire procedure can be performed with little or no waiting between stages.

Centralizers are normally employed to assure concentric alignment of the pile in the drilled hole. Spring or bow type centralizers are generally preferred because they give less interference when running the pile in the hole and give better centralization than solid bar type centralizers. In addition, properly designed shear rings or lugs can be welded to the pile wall to ensure effective shear load transfer at the pile grout interface especially in the pile sleeve connection area (Fig. 4-1).

4.2.2 Grout Line Method

This method is common in mine shaft casing grouting. Grout is injected through a grout line which is a small-diameter pipe or tube welded to the outside of a pile or casing. Several strings of grout line may terminate at various depths for grouting in several stages.

Alternatively, one or more retrievable grout lines may be run into the pile borehole annulus after the pile is in place. Grout is pumped and the grout line is pulled out. Grouting may be performed in several stages by pulling the grout line(s) to the calculated (or measured) "top of grout," circulating contaminated grout out of the annulus, and pumping successive stages of grout after the previous stage has set.

4.2.3 Delayed Set Method

The delayed set method consists of running a grout line to the bottom of the borehole before the pile is placed. A retarded grout is pumped to the bottom of the drilled hole and the grouting line is removed. Then the pile is lowered into the grout to total depth. Subsequent stages may be performed through grout lines in the pile borehole annulus before final grout setting, as previously described.

4.3 Grout Types and Properties

For drilled and grouted piles, the primary concern in the selection of grout type is strength development and shear bonding. High strength grouts are not necessary since the grout strength need only be higher than the shear strength of the adjacent soil for maximum shear transfer.

Development of grout strength is basically a function of two factors: (1) additives which accelerate or retard setting time, thus affecting early strength, and (2) the water/cement ratio which affects both early and final strengths. Final strengths can also be increased by the addition of silica sand or flour. A low water/cement ratio, which produces higher strengths, requires more materials, thus is more costly. It also results in a high density grout which may not be desirable when hydraulic fracture is a problem. Therefore, grout formulation design is generally a compromise between strength, density, and cost.

4.3.1 Low Density Filler Grouts

These formulations have low densities which range from 12.0 to 13.0 lb/gal (90 to 95 lb/ft^3). They generally incorporate an extender such as bentonite, and employ high water/cement ratios. They exhibit poor early strength development and low final strength which ranges from 1,000 to 2,000 psi.

Low strength filler grouts are often used where higher density grouts would cause hydraulic fracture or lost circulation problems. They are also used where minimal strength is required, as for example, when filling the inside of piles.

4.3.2 Light-Weight Maximum Strength Grouts

These grouts, which are formulated using "high strength microspheres", exhibit superior compressive strength compared to conventional filler grouts at equivalent density. Their density ranges from 10 lb/gal (75 lb/ft^3) to about 13.5 lb/gal

(100 lb/ft^3) with final compressive strengths ranging from about 500 to 5,000 psi, respectively.

4.3.3 Moderate Density - Normal Strength Grouts

Moderate to normal strength grout formulations, with densities ranging from about 13.0 to 14.2 lb/gal (95 to 105 lb/ft^3) generally offer a reasonable compromise between strength and cost. Most drilled pile annulus-to-formation grouts fall within this type, with final strengths from 1500 to 3500 psi when conventional extenders (such as bentonite) are used. Lightweight additives can be used which give final strengths up to 6000 psi.

4.3.4 High Strength Grouts

These formulations vary from neat cement (normal water requirements) with a density of about 15.5 lb/gal (115 lb/ft^3) to minimum water, high sand grouts at 18.5 lb/gal (140 lb/ft^3). Final strengths are typically from 7000 to 12,000 psi. These grouts are most commonly used for pile-template or pile-jacket connections. However, they could have some application for drilled and grouted piles in hard rock. Shear rings would be needed in order to realize the high load transfer potential.

4.4 Grout Additives and Properties

4.4.1 Accelerators and Retarders

Accelerators are used to hasten the initial set time and to increase the early compressive strength. Accelerators do not increase the long term strength of grout.

Retarders are used to delay the initial setting time. The ultimate grout strength is not affected by these additives. The use of retarders to delay set time may be advantageous for drilled and grouted piles in deep water since long pumping times may be required in order to place large quantities of grout at the desired depths.

4.4.2 Fluid-Loss Control Additives

Grout formulations may include fluid-loss control additives (also called filtration control or water retention additives) to reduce the rate of water loss to permeable formations. Water loss during the time the grout is in a plastic state increases grout shrinkage and reduces both the formation-grout and the pile-grout bond strength.

4.4.3 Bridging Materials

Bridging materials are often used to help control loss of grout which may be due to hydraulic fracture or highly permeable soils, such as cavernous limestone or gravel. These additives include gilsonite, walnut hulls and coarse sand. They can be used in grouts of all densities and are often included as a preventive measure when grout density is close to the hydraulic fracturing gradient of the formation or when gravel or other coarse materials are known to be present.

4.4.4 Expansive Additives

In general, all grouts shrink slightly as they change from a pumpable mixture to a set solid. Expansive additives cause overall expansion of the set grout. Expanding grouts can be prepared at all densities. The formulation varies with the type of cement and the amount of expansion needed.

Plastic state expansive additives can be used to compensate for water loss and shrinkage (due to hydration reactions) while the grout is in its plastic state. These additives generate gas in situ and effectively increase bonding to both the pile and the formation. This approach is useful where high bond strength is necessary.

4.5 Grout Preflushes

Preflushes are generally not required when grouting drilled piles since low viscosity drilling fluids, which usually occupy the annular space, can be easily displaced with grout. However, a great deal of positive experience has been accumulated with reactive preflushes pumped ahead of the grout. These reactive preflushes are very effective in removing borehole fluids, reducing water loss to permeable formations, reducing lost circulation and improving grout-to-formation bond. Therefore, their use in constructing drilled and grouted pile foundations may be worth considering more closely.

4.6 High-Pressure, Multiple-Injection Grouting Technology

The high-pressure, multiple-injection grouting (HPMIG) technique has been developed and patented by Soletanche of France. This technique has been used extensively onshore for tie-back anchors, short drilled and grouted piles, grouted curtains and compaction of loose materials (soil reinforcement). Recently, the method has been applied offshore for re-installation of a well casing in unstable permafrost soils. Also, the technique was used to regrout the bottom portion of an instrumented drilled and grouted test pile installed and load tested at a site in the Gulf of Mexico (Mueller et al., 1986).

4.6.1 Grout Placement and Procedures

Pressure grouting originated approximately 50 years ago with the Soletanche patent of "Tubes à Manchettes" (TAM) or "sleeve-tube" grouting. Originally developed for grouting alluvium deposits, this technique was soon adapted to increase the pullout capacity of tie-back anchors (a Soletanche patent known as the IRP anchor). The technique has been further modified for application to cast-in-situ piles as well as drilled and grouted piles. Most of today's experience related to pressure grouting still relies on a combination of experience, theoretical analyses and test results.

<u>The TAM Technique</u>. The TAM technique (Fig. 4-2) was developed to impregnate alluviums with grout in order to reduce their permeability (as in dam cut-off wall applications) or improve their engineering characteristics. The principle of the system relies on the use of a double-packer inserted inside a small pipe which is perforated at regular spacings. The perforations are covered with rubber sleeves (Fig. 4-2) which act to restrict the return flow of grout back into the grout pipe once pumped into the formation.

The grout pipe is lowered into the previously drilled borehole. The primary grout is then pumped through the lower sleeve to fill the whole annulus as performed in any conventional gravity grouting job. The grout is then allowed to set.

A packer is used to reinject grout under high pressure at any preselected depth by allowing the new grout to hydraulically fracture the previous layer, or primary jacket of grout, forcing new grout out and around the previous grout.

<u>The IRP Anchor Technique</u>. The TAM technique was later applied to soil anchors (IRP anchors) in order to increase their pullout capacity by (1) forming a grout bulb which is much larger than the initial borehole diameter, (2) improving of soil characteristics in the vicinity of the bulb due to the impregnation or squeezing of the soil (depending upon whether or not the soil is permeable to the grout), and (3) improvement of the grout characteristics by squeezing out water in the pore spaces of the soil in the borehole wall.

The effect of high-pressure, multiple-injection grouting (HPMIG) has been studied and observed by the extraction of grouted piles and soil anchors. By using grouts of different colors, the mechanism of successive phases of grouting could be studied as shown in Figure 4-3. These studies indicate that (1) the mean diameter of the bulb can be closely related to the quantities of grout placed, provided hydraulic fracture is prevented, and (2) the failure zone is generally outside the grout-soil interface which is possibly due to an improvement of soil properties and to the irregular surface of the grout bulb that forces failure along an overall outside surface.

<u>Toe-Grouting</u>. This technique has been used for cast-in-situ concrete piles, where the hole has been drilled with a drilling fluid composed of bentonite and then tremie

concreted. Toe-grouting involves grouting around the pile shaft through a pipe (2-in. diameter typically) installed inside the pile (Fig. 4-4). The pipe is plugged with plaster or a rubber cap during pile casting.

Three bored piles were constructed, load tested and extracted in Dunkerque, France (Fig. 4-5). Figure 4-5a shows the two grout pipes exiting at the tip of one of the three piles. In Figure 4-5b one can notice the grout film formed by pressure grouting. The grout film was intentionally broken off the pile after extraction in order to better visualize its thickness and uniformity. Inspection of the piles after extraction disclosed the following:

1) A very uniform spread of the grout occurred along the length of the pile (grout thickness of 0.3 to 0.4-in.).

2) The total quantity of grout injected (22 gal.) corresponded closely to the volume of the grout coating around the pile.

While rising of the grout along the shaft has been demonstrated to work along lengths of approximately 30 to 50 ft., this procedure is not well verified for larger diameter and longer piles. Thus, further field testing is recommended.

<u>Shaft Grouting</u>. This is an alternative technique for grouting cast-in-situ piles. A pressure grout is applied through "tubes à manchettes" (TAM) placed into small boreholes drilled in close proximity to the pile after the pile has been concreted, as shown in Figure 4-6 (Bustamante et al., 1983). Typically, a 1.6-in. plastic pipe (with perforations and rubber sleeves) is placed inside a 2.4-in. drill hole which is then filled with grout under gravity. It is then possible to pressure-grout successively at all depths, and if necessary, to apply several stages of grouting.

This technique is limited to relatively short pile lengths due to the difficulty in minimizing deviations associated with drilling small boreholes. Taking into account conventional deviations for such holes (e.g., 1 in 50) and assuming that the grout hole should not be further than 8-in. from the piles, this technique would appear to be limited to pile lengths of approximately 80 ft, unless much more sophisticated controls on borehole deviations are used.

<u>Central Double-Packer System</u>. This system was developed for drilled piles with inserts where the hole is drilled by conventional means using water or bentonite mud. When drilling is completed, an insert pile whose wall has been fitted with reusable non-return valves, is lowered into the hole. A central double-packer system of the size of the insert is then lowered inside the insert pile and pressure grouting started. The reusable non-return valves allow several phases of grouting to be performed at any desired elevation.

This method requires the use of very large double packers which are not readily available on the market or may not be compatible with the use of low water-cement ratio grouts. This technique was used with a fair degree of success on the Conoco instrumented test pile (Mueller et al., 1986). Its feasibility should be further investigated so as to be perfected for deep water applications.

A swab or seal cup packer tool has also been used for grouting the annulus between foundation piles and jacket sleeves. The tool consists of two pairs of inverted rubber cups spaced at a distance of one to two feet from a central cement port. The tool is lowered through the insert pile and stopped at the location of grout ports located in the wall of the insert pile. The ports may be preinstalled, one-way grout values or the casing may have been perforated following running into the borehole.

Once the grout port is isolated, grout is pumped through the drill string to the cup packer. As pressure between the cups increases, the cups expand and seal against the internal diameter of the insert pile. Grout is then forced out the grout valve or perforations in the casing into the annulus. This technique was also used with a fair degree of success on the Conoco test pile. Its feasibility should be further investigated so as to be perfected for use in deep water.

<u>Central Grouting System</u>. The central grouting system shown in Figure 4-7, consists of a coaxial 7-in. diameter pipe preinstalled inside the insert pile. This inside pipe is connected to the outside wall of the insert by non-return valves placed at regular spacings along the insert. A 6-in. double-packer system is run into the 7-in. central grouting tube to isolate, one by one, each of the non-return valves, allowing multi-level, multi-stage grouting (Fig. 4-8).

The 6-in. packer can be set on flexible hoses or on cables. It is fitted with:

1) A water line to inflate or deflate the packers,
2) A second water line for washing out the grout, and
3) A grout line to inject the grout.

Although the above procedures should be further adapted and developed for deep water TLP use, they can be generally considered to consist of the following phases (Fig. 4-9):

Phase A – Primary Cementation: The pile is lowered down through the template and into the borehole and primary cementation is performed in the conventional way with a stinger locked into the casing shoe.

Phase B – Reentry of double packer set: After completion of primary cementation, the double packer set is lowered and introduced into the coaxial tube. The packer is connected to the surface by a hose bundle which includes lines for inflation and deflation of the packers, cementation, washing and operational control. Note that such reentry is done only once per pile as all of the operations, at successive levels, are performed while the double packer set remains in the pile.

Phase C – Primary cementation integrity check (Case 1): The double packer set is first positioned at the lowest grouting box level. Special controls ensure the exact positioning before the packers are inflated. The test begins when water is pumped through the cementation line, and the variation of pressure with time is recorded. The test is stopped when the pressure reaches a value, P_m, which is high enough to ensure that the hardened cement prevents the valve from opening, but is low enough so that cement is not fractured. Packers can be immediately positioned at the next upper level and the procedure of primary cementation integrity check repeated.

Phase C – Primary cementation integrity check (Case 2): The test procedure is the same as in Case 1 above except the annulus is found to be filled with fluid directly connected with the seafloor surface. In this case, the forces which prevent the valve from opening are only caused by the hydrostatic pressure in the annulus and the nominal opening resistance of the valve membrane. Therefore, as shown in the insert of Figure 4-9 for Case 2 at Phase C, the pressure increases gradually until the valve opens and lets water flow into the annulus under a more or less constant pressure close to the hydrostatic pressure P_o. Additional primary cementation is thus determined to be required at that level of the pile (Phase D).

<u>Phase C - Primary cementation integrity check (Case 3)</u>: The test procedure is similar to that used in Case 1, however, in this procedure, the pressure may drop suddenly when it reaches a value much smaller than would have been necessary to fracture the set grout but higher than the hydrostatic pressure, P_o, measured in Case 2 above.

This behavior would indicate that the fracture occurs in very weak cement or directly in the soil. The cause may be the presence of mud pockets trapped in the cement or a direct contact between pile and soil. In any case, this indicates that additional cementation is needed at this particular level (see Phase D).

<u>Phase D - Additional cementation</u>: As shown in Figure 4-9, cement is pumped through the cementation line and pushes out the water used for the test (Phase C). The pressure is controlled so that the valve stays closed and water does not flow into the annulus. The pressure is then increased to open the valve when cement reaches the level of the valve through which additional cementation is required.

Pressure variations are recorded during cementation as sudden changes may indicate soil fracture or soil collapse. This also gives valuable information on the filling operation and on the time when cementation should be stopped. Once grouting is completed, the entire grouting system is flushed in place so that it can be immediately used again.

Every re-cemented level must be checked again after setting and eventually re-cemented until final check shows the presence of hard cement everywhere along the pile length.

For developing high pressure grouting techniques, the Soletanche group has developed various pieces of grouting hardware among which two are most relevant to this study. These are the MGBH and the BHPC System, as discussed below.

<u>MGBH System</u>. The MGBH system (More Grout in the Bore Hole) is a grouting technique that can be used when access to the inside of the insert pile is not possible (Barthelemy, 1978a). It consists of a non-return valve which is set on the outside of the pile wall and is connected to the surface by a double line (Fig. 4-10). A number of MGBH systems can be placed along the pile length. Each MGBH is then used to perform multi-stage grouting. The principle of the system allows for:

1) Checking of primary cementation,
2) Measuring the pressure during grouting,
3) Reducing the hydrostatic head in the soil (avoid fracturing), and
4) Performing multi-stage grouting.

Dome Petroleum recently used the MGBH system in the Canadian Beaufort Sea to reinstall a well casing through unstable permafrost soils. This equipment may be used if a limited increase in frictional capacity is needed, or to make certain that a proper primary cementation has been performed.

A modification to the standard MGBH system was developed for Woodside Petroleum for work in their Rankin field, offshore Australia. A junction manifold would be used to collect all of the return lines from each MGBH unit at a position just above the upper most unit (Fig. 4-11). Each individual return line, terminating inside the sealed junction manifold with a TAM-type one-way valve, would only allow water or grout flow in one direction. By back pressuring the common return line (P_W) to a pressure above the grout pressure being applied to one of the grout lines (P), the TAM-type one-way valve inside the junction manifold would be forced shut and grout would be forced to exit from the designated MGBH unit along the pile wall. Flushing of the system would be performed by releasing the return line back pressure and pumping seawater through the open lines until no further grout appeared at the end of the common return line.

BHPC system. The BHPC (Bore Hole Pressure Control) system (Fig. 4-12) has been designed to obtain a continuous and simple recording of the grouting pressure around the annulus of a pile (Barthelemy, 1978b). This simple and reliable system is well suited for casing or pile cementation purposes with no electronics nor sophisticated equipment involved.

4.6.2 Quantitative Effects of Pressure Grouting

Effects of pressure-grouting on small-diameter piles and soil anchors was reported by Gouvenot (1973). The skin friction, f_s, along the inferred grout bulb surface (computed from the grout quantity pumped) was shown to be of the same order of magnitude as that obtained by primary (gravity) grouting alone although generally higher. Some of the results are shown below:

Calculated skin friction from experimental results	Sand-Gravel	Soil Type Sandy-silts	Clay
Unit friction for gravity grouting (tsf)	0.8 to 1.0	0.4 to 1.3	0.25 to 0.5
Unit friction along the inferred bulb diameter after pressure grouting (tsf)	2.2 to 3.0	0.4 to 2.30	1.0 to 1.5

4.7 Quality Control

The construction of a drilled and grouted pile at a deep water offshore site poses many uncertainties due to the depth of water involved making the task a "remotely sensed" operation. Borehole verticality, borehole diameter and general configuration can be determined from borehole caliper runs performed immediately after drilling.

The configuration and the uniformity of the grouted annulus around the insert pile is what will ultimately determine the load-carrying capacity of the tension pile. Inspection of the grout's (and hence, pile's) integrity will be crucial in order to allow for loading of the pile by the TLP.

Quality control measures can be divided into those that are performed during grouting and those performed after grout set. Monitoring of the grout during the grouting operation may include (1) density checks of the grout with a radioactive densometer, (2) monitoring of the top of grout during cementing, and (3) monitoring grout pressure downhole. Quality control measures taken after the grout sets include (1) stress-strain and strength tests of the grout samples, and (2) running of logging tools into the casing to obtain information on pile-grout and grout-formation bonding, grout integrity, and possibly even grout geometry. These methods are discussed in subsequent sections.

4.7.1 Radioactive Densometer

The radioactive densometer was originally developed by Halliburton Services to monitor in-place grout density and is presently marketed by Wimpey (Fig. 4-13). The grout density is measured in a cell as the slurry passes through a flow tube located between a radioactive source and a radiation detector. This instrument can measure the density of slurries in any composition which contain elements up to and including atomic number 56 (barium). The tool is capable of measuring densities in the range of 8 to 20 ppg.

4.7.2 Grout Sampling

Sampling of the grout as it leaves the annulus at the seafloor has been performed with diver assistance in shallow water. For deep water operations, a remotely operated vehicle (ROV) may be used. However, it is questionable whether this sample could be obtained and transported to the deck intact and before the grout sets. Thus, the most likely alternative would be to sample the grout from the discharge line on deck. Grout viscosity and density can be checked and samples made for strength measurement.

4.7.3 Grout Position Monitoring

Monitoring of the grout position in the annulus is important in order to know whether problems are occurring such as excessive grout take due to hydraulic fracturing or because a large cavity is being filled. In the past, radioactive particles have been added to the lead portion of the grout. Gamma logging tools have been run into the pile or within small grout tubes to track the position of the grout in the annulus (Callis et al., 1979). This method appears feasible for TLP piles provided the use of radioactive material is permitted. Alternative systems include retrievable temperature or resistivity sensors run on an electrical umbilical in the annulus or pressure sensors permanently attached to the outside of the pile to measure the head of grout rising in the annulus.

4.7.4 Grout Pressure Monitoring

Pressure transducers to measure grout pressures in a cemented annulus have been previously attempted as described by Cooke et al., (1982). Solmarine has also developed two systems that could be used for such an application. These are the MGBH and BHPC systems as discussed in Section 4.6. The MGBH system developed by Solmarine may also be used to measure grout pressure downhole during high pressure grouting (Fig. 4-12).

4.7.5 Cement Logging Tools

A number of downhole tools have been developed to log the annulus of a surface casing when cemented or grouted in place. This equipment is typically used to (1) determine the top of grout after primary cementation has been performed, (2) the quality of the steel-cement bond and (3) to qualify the integrity of the cemented annulus e.g., whether channeling or large voids are present. These tools, commonly called cement logging tools, have generally not been used to log casings greater than 13-in. diameter. Research and modifications to the tools and/or their theory is thus necessary to upgrade these systems to meet the industry needs to log larger diameter and greater wall thickness casings. The cement logging tools available all use the principle of sonic wave propagation, and can be classified into refraction and reflection type.

Refraction tools, such as Schlumberger's **Cement Bond Tool** (CBT) and **Cement Bond Log** (CBL) (Fig. 4-14) generate short pulses of high frequency signal (20kHz) directed radially through the casing into the borehole. These signals travel down the casing walls, grouted annulus and soil formation. Refracted pulses are then picked up by the receiving transducer of the CBT. Quality of grout-casing bond can be estimated (Gollwitzer and Masson, 1982). The bond strength can be estimated provided the bond quality is high. Careful centralization of the transmitter and receiver transducers is essential to minimize spurious amplitude variations from unequal-path length interference.

Schlumberger's **Cement Bond Log** (CBL) also incorporates a variable density log which records the head and surface waves (when present) traveling in the formation (Brown et al., 1970). Schlumberger provides guidelines for the analysis of the formation-wave amplitude variations to obtain a semi-quantitative estimate of the strengths of the cement grout and the grout formation bond. Both refraction tools do not provide a measure of formation distance or casing thickness, but may give indirect indication of inclusions, voids, or fractures within the grout.

Reflection tools detect single-and multiple-reflected waves by a transmitter/ receiver transducer (Fig. 4-15). In contrast to the axially symmetric refraction wave propagation, focused sonic waves for the reflection tools are confined to a small angular interval and travel radially. Much higher frequencies, such as several hundred kHz, are used for most of the available reflection tools in order to enhance resolution.

Schlumberger's **Cement Evaluation Tool** (CET) utilizes eight transmit/receive transducers examining different azimuths in cased boreholes (Froelich et al., 1981). For each azimuth, the reflected wave train includes primary reflections from the casing, grout, and formation boundaries, and a decaying echo train from multiple reflections in the casing wall. The amplitudes of the reflected waves are a function of bond integrity and the acoustic impedance of the constituent material. The analysis of appropriate regions of the reflected waveform provides a more detailed description of the grout/bond/formation behavior than the cement bond tool. The transmitted frequency in the CET is designed to maximize the amplitude of the multiple-reflection wave train which is much larger than the primary reflections. The tool does not yield the formation-reflection arrival time (and hence a measure of grout thickness), however.

4.7.6 Other Grout Logging Tools

Several other tools and systems exist or are being developed to log the grouted annulus of a surface casing or a drilled and grouted pile. They are as follows:

1) Borehole Televiewer (BTV)
2) Pulsed Neutron Capture (PNC)
3) Nuclear Fluid Density (NFD)
4) Gamma-Spectral
5) Radial Differential Temperature (RDT)
6) Seismic
7) Ultrasonic Sonde

The **Borehole Televiewer** has been used to inspect the condition of uncased boreholes. A rotating transducer transmits and receives signals from the formation wall. If such a tool were to be used to log drilled and grouted piles, modifications would be necessary to accommodate the particular environment.

The **PNC, NFD** and **Gamma-Spectral** systems are nuclear logging tools conventionally used for delineating soil formations as with the borehole televiewer above. Due to their ability to measure radiation density, these tools could be used to measure grout thickness (and, hence, volume) and top of grout if a radioactive tracer were mixed with the grout. Again, this technique would require some development time to adapt to logging of drilled and grouted piles.

The RDT would also require some calibration, and possible modifications, in order to relate the amount of heat released by the exothermal reaction of the setting grout to the thickness of grout being monitored.

The use of conventional seismic techniques may show considerable promise due to the ability of using an unlimited source from outside the pile interior. A downhole receiver could be positioned at various depths and azimuths to obtain directional data of grout integrity or to record an average degree of grout quality around the pile circumference.

Grout logging tools are available from several sources. During our search we investigated the following sources:

1) Schlumberger Offshore Services
2) Amoco Production Company
3) Chevron, USA
4) Gaz de France/IFP

Early discussions with the first three groups above indicated that Schlumberger was the only commercial group in a position to provide immediate cement logging services for drilled and grouted piles. Amoco and Chevron both operate tools developed by Schlumberger and which are being subjected to calibration trials under an advanced research and development program.

Schlumberger has been involved in a full-scale laboratory calibration of their cement logging tools in anticipation for the need to log well casings and drilled and grouted piles of larger diameter and greater wall thickness than used today. Onshore tests on 13-3/8-in, 20-in and 30-in OD casings cemented into the ground have been performed using sonic and nuclear tools ex-centered from the centroid of the casing. Such tests showed promise although the 1-in wall thickness on the 30-in casing could not be penetrated easily. Ex-centering of the CET and the NFD tool revealed that 0.5-in wall casing could be penetrated successfully. Schlumberger has used downhole cement logging tools offshore on casings as large as 20-in OD with a reasonable degree of success.

The French Petroleum Institute, in collaboration with Gaz de France and Elf Aquitaine, has developed the **ARTEP Ultrasonic Sonde.** This probe has been successfully used to qualitatively measure the condition of the grouted annulus in a cased hole. Direct comparison of data have been made with the CBL, revealing that data from the Ultrasonic Sonde is in general agreement with that from the CBL although of higher quality. Presently available information as to the casing diameter capacity of the tool indicates that maximum casing size is of the order of 20-in OD. Commercial use of the probe outside the French government agencies appears to be a stumbling block, however.

4.8 Technology Gaps in Grouting

Several potential problems associated with grouting of drilled and grouted piles exist. Grout requirements for the Thistle A and Piper platforms were typically 60 percent more than planned. In one particular pile case, more than 300 percent of the planned volume of grout was required. Grout was also found in an adjacent pile hole which suggests that hydraulic fracture occurred. Other significant concerns have also been identified from the experience gained in grouting offshore wells and must be addressed for TLP piles.

Primary Grouting in Deep Water. The potential effects of primary grouting in deep water for all soil types include:

1) Hydraulic fracturing leading to excessive grout loss and possibly reduced load transfer due to lower grout pressures, and

2) Pile flotation, due to buoyancy forces, leading to inappropriate pile installation.

In deep water situations, especially in soft or loose surficial sediments, even primary grouting under gravity may cause hydraulic fracture due to the high column of grout pumped into the pile annulus. The resulting loss of grout makes predictions of grout quantities difficult. Also, predictions of pile load capacity become unreliable due to incomplete and undefined grouting of the pile annulus.

Identification of the hydraulic fracture gradient of a particular soil formation is dependent upon the soil permeability, mud viscosity, lateral bearing capacity of the borehole wall and the presence of microfissures or seams of different soils. The hydraulic fracture gradient can be defined by the following methods:

a) Borehole packer tests. A double packer system, which isolates a specific interval of the borehole wall, would be more preferable to a single packer system since it would define the fracturing zone more exactly. The lateral mud pressure is applied directly to the borehole wall only. Fracturing originates directly from the wall in either the vertical or horizontal direction depending on the state of stress in the soil and whether the soil is normally consolidated or over-consolidated.

   Microfissures, due to stress relief from the drilling operation, or planes of weakness due to seams or layers of cohesionless soils may initiate premature fracture of the formation upon intrusion of the drilling mud (or grout during cementing operations) along such contact weaknesses. The mudcakes formed on the borehole wall during drilling could help block such microfissures, but it may reduce the load transfer of the pile as well. In view of the above conditions, packer-type techniques may indicate a lower bound measurement of hydraulic fracturing.

b) Pressuremeter tests. The technique has long been used to determine the limit pressure of subsurface soils. A single packer-type tool is inflated against the borehole wall. The inflation pressure and volume are monitored until shear failure of the formation occurs as indicated by unlimited volume increase at constant pressure. The pressuremeter evenly distributes the inflation pressure over the surface of the borehole wall without allowing fluid intrusion into microfissures or seams. Hence, under equivalent conditions, the pressuremeter would most probably register a greater hydraulic fracturing gradient than that recorded from the single or double packer-type system described above. The pressuremeter technique is thus felt to provide an upper bound measurement of the hydraulic fracture gradient.

c) Analytical techniques. Hydraulic fracture gradient of a formation can be predicted using equations 3-1 and 3-2 for cohesive soils and equations 3-3 and 3-4 for cohesionless soils. As previously mentioned, it is doubtful that these simple equations, which contain only a few key parameters, would take into account all factors associated with hydraulic fracture. Further study is necessary to develop more reliable analytical method to estimate the hydraulic fracture gradient.

The phenomenon of hydraulic fracture due to grouting is similar to that associated with drilling of the borehole using drilling muds. Typical solutions for the problems are: (1) using high flow rates in the annulus with reduced pressure, (2) using light-weight grout, and (3) using multiple stage grouting.

Grouting in deep water could potentially lead to pile flotation due to the difference in fluid densities inside and outside of the insert pile during grouting. This has been found to be particularly true for large diameter piles (Taylor, 1986). Proven procedures are available for controlling this problem. In addition to using the remedial measures identified above for reducing grout pressure, consideration should also be given to filling the insert pile with a heavy fluid, latching the pile into the template until the grout sets or performing a multiple stage cement job.

Excessive Grout Loss. Grout loss causes difficulties in controlling and predicting grout quantity. Potential remedial measures include (1) the identification of problem soils, (2) the estimation of required amounts of grout, and (3) the use of bridging materials (Section 4.4.3) to minimize grout seepage into the formation. All of these could be done during the site investigation and the exploratory well installation. Solutions can then be developed during preliminary design.

Another possible remedial measure would be to monitor the grout position in the annulus space using existing technology. Research and development in this area should be considered.

Mudcake Considerations. A thin mudcake can form on the sides of the borehole during drilling. If the mudcake is not removed prior to or during grouting, reduced load transfer could potentially result. Possible remedial measures include (1) drilling with seawater and gel to avoid mudcake formation, (2) use of grout pre-flushes or mechanical scraping to remove mudcake, and (3) use of longer piles to compensate for the uncertainty in shear transfer. Alternatively, multiple high pressure grouting could possibly be used to consolidate the mudcake and to create an irregular pile geometry and thus force potential shear surface outward into the soil mass.

Quality Control. Quality control both during grouting and after the grout has set is extremely important to ensure proper installation of drilled and grouted piles. Additional information will need to be gathered during pile installation to verify (1) full displacement of the drilling fluids, (2) the configuration of the grout jacket, (3) that hydraulic fracture has or has not occurred, and (4) whether remedial measures are needed.

Multiple high pressure grouting should be further investigated and seriously considered as a back-up grouting plan as it offers the capability to perform remedial grouting at the time of pile installation or even afterwards. Useful information on grouting problems, types, and procedures should be gathered during well installations.

Remedial measures will also involve monitoring of the grouting operations. Existing tools will have to be modified or new equipment developed to (1) run through large diameter insert piles to determine grout geometry and presence of voids, mud pockets, and soil cave-ins and (2) monitor grout pressure and grout position in the annulus. Testing of existing tools is felt to be a priority. Grout pressure monitoring tools would help to provide the necessary information as to whether hydraulic fracturing has occurred or may soon occur. Further work is recommended to understand the magnitude of allowable grout pressure as a function of soil properties and hydraulic fracturing pressure.

5.0 ANALYSIS AND DESIGN METHODS

5.1 General

A literature review of the methods used to estimate the capacity of drilled and grouted piles indicated that empiricism, intuition and past experience play an important role due to the complex behavior of a) the composite pile cross-section and b) the soil-grout interface under load, particularly in tension. For TLP applications, the problem is further complicated by the difficulty in obtaining soil properties in deep waters, complex loading conditions, as well as construction and quality control problems. Various factors to be considered in the design of drilled and grouted piles include the following:

1) Soil conditions
2) Drilling procedures
3) Grout types and grouting techniques
4) Pile installation techniques
5) Pile configurations
6) Grout behavior under static and cyclic load
7) Behavior of interface bonds under static and cyclic load

Only limited information is available on the behavior of drilled and grouted piles, especially under cyclic loading, thus experience gained from bored piles, drilled shafts, piers and soil anchors is commonly used to assess the capacity of offshore drilled and grouted piles. Pile capacity from such sources likely represents an upper bound because of the better working conditions and quality control associated with onshore work.

5.2 Soil Conditions

The site characterization procedures at deep water locations usually incorporate the following three steps:

1) Geological survey. This is a large scale horizontal-dimension survey which involves the evaluation of historical records of geology, bathymetry, sediment types and origin, climatic changes, and other environmental factors that may affect characteristics of the foundation soils.

2) Geophysical survey. This is a large scale vertical-dimension survey to define bathymetry, seafloor structure as well as subseafloor layering and structuring. Geophysical tools which are normally used for this purposes are (a) high-resolution seismic reflection systems, (b) acoustic profilers, and (c) side-scan sonar systems. Each of these instruments provides its own unique set of information. It is therefore desirable to use them in combination.

3) Soil investigation and testing. This includes soil sampling, laboratory tests and in situ tests. The objective is to provide the foundation designer with the necessary soil parameters.

The steps described above yield the necessary input for analytical models used to predict future soil response in association with drilled and grouted piles. More complete treatments on the above subjects can be found in the works by de Ruiter and Richards (1982), Quiros et al (1982), Tjelta et al (1982), Campbell (1984), Lee (1984), Richards and Zuidberg (1984), Templeton et al (1985), Aas et al (1986), Campbell et al (1986), and Peterson et al (1986).

5.3 Failure Mechanisms of Drilled and Grouted Piles

5.3.1 Failure Surface

Under tension, drilled and grouted piles may fail under one or a combination of the following models:

1) Grout-Soil or Soil-Soil Interface Failure. This is the most likely failure mode to occur. Section 5.4 will review current methodologies used to

estimate the shear resistance along or near the soil-grout interface. Relevant pile load test data are also discussed.

2) Steel-Grout Interface Failure. This mode is uncommon since the shear resistance at the steel-grout interface is generally much higher than that at the grout-soil or soil-soil interfaces. However, when the shear resistance along or near the grout-soil interface becomes large such as in the case of piles in heavily overconsolidated clay or when the grout layer is thick and irregular, this failure mode may need to be checked.

3) Failure of the Grout Layer. Even though Cox and Reese (1978) indicated that this mode is uncommon in normal applications, it should be further investigated in a case of TLP foundation piles due to the tensile and cyclic nature of the loading. Tension cracks in the grout may form due to incompatibility of the stress-strain behavior of the steel and grout.

5.3.2 Load Transfer Behavior of Grout

Pile-Grout Bond. The bond between casing (pile) and grout can be classified as hydraulic, gas, and shear bonds. Hydraulic and gas bonds relate to the bond strength which is capable to withstand hydraulic and gas pressure without leakage. They are of little interest to the design of drilled and grouted piles. Shear bond relates to the pile-grout shearing resistance, which is important in pile applications.

Figure 5-1 shows the test results of the pipe-grout bond strength as a function of pipe surface finish, grout curing time, and surface wetting of pipe. A standard cement grout with a water/cement ratio of 0.46 was used in all tests.

Table 1 in Figure 5-1 shows the bond strength between different types of surface finish after one day of curing time at 80°F. The lowest shear bond strength (74 psi) was found for a new mill varnish surface and the highest (2400 psi) for a surface with resin-sand coated after sand blasting. A picture of the different pipe surfaces tested is shown in Figure 5-2.

Table 2 in Figure 5-1 shows the bond strength of the grout at different curing times. New mill varnish surfaces were used and the grouts were all cured at 100°F. Within a five-day period, the shear bond strength seems to remain constant while the hydraulic and gas bond strengths increase up to 35 and 350 percent, respectively.

Table 3 in Figure 5-1 shows the shear bond strength variations for different surface wetting of a used (rusty) pipe. This shows that the bond strengths vary from as low as 63 psi for an oil-based surface wetting to 97 psi for a water-based wetting, and 141 psi for no wetting. These results illustrate the importance of preflushing to remove drilling fluid from the pile surface and improve bond strength.

A shear bond strength value of 20 to 35 psi is commonly used in practice when there are no supporting data (Kraft et al., 1974; Ehler et al., 1977). For a plain pipe, API (1986a) suggests an allowable bond strength of 20 and 27 psi for normal operating conditions and design environmental conditions, respectively.

Figure 5-3 presents the results of laboratory experiments on steel-grout bond strength for different grout types as reported by Sol Expert (1973). According to the test results, the bond strength can vary from as low as 430 to 580 psi for cement grout with a water cement ratio of 0.4 to as high as 2,200 to 2,750 psi for epoxy resin grouts.

<u>Grout Behavior During Cyclic Tension Loading</u>. Surface casings are usually run to 100 ft penetration or more. The bending stresses from lateral loads are not a significant design consideration since they occur in the confined cement annulus. Therefore, the following discussion will focus only on the grout behavior under axial loads.

The axial load-displacement behavior of a drilled and grouted pile under cyclic loading is difficult to define because of the composite nature of the pile cross-section (steel, grout annulus, and soil). Due to major differences in material properties, the imposed strain level will force the materials to perform differently after many cycles of load rather than act as a composite section. Even at low strain levels under cyclic loads, stress redistribution between steel and grout could occur due to (a) creep effects, whose degree may depend on a percentage of cyclic component versus bias load, and (b) shrinkage of grout, which depends on different formulations and curing methods.

Figure 5-4a shows the conceptual behavior of a composite section under tension. Upon initial loading, it is anticipated that steel and grout will act together as a composite material as long as the strain level is small and is within the elastic region of each material. With continued tension (static and cyclic) creep in cement would eventually lead to microcracks and loss of continuity in the grout. Similarly, at higher load, cracks start to propagate along the length of the grouted annulus. This gradual breakdown of the grout integrity causes a reduction of composite stiffness, and finally the steel will carry most (if not all) of the load. It is possible that a more instantaneous breakdown of the grout occurs (Fig. 5-4b) and a much more rapid transition may take place from the behavior of the composite section to that of the steel alone.

Therefore, the initial and final stiffnesses of the composite section can be bound. The transition between these two extreme cases is difficult to estimate for any given point after initial loading and may best be resolved through experimentation.

For a drilled and grouted pile, cracking of the grout jacket under cyclic tension is not well understood. Spalling of the grout could progress downward starting at the upper portion of the pile where the section is most strained. Because the lateral pressure of the surrounding soil would maintain the spalled grout around and against the insert pile, it is felt that perhaps only a portion of the shear transfer capacity of the pile would be lost, and the spalled grout may possibly provide some stiffness to the pile as well. Again this may best be resolved through experimentation in the lab or preferably in the field.

5.4 Axial Behavior

5.4.1 Ultimate Axial Capacity in Cohesive Soils

With a grout-soil (or soil-soil) interface failure mode, the ultimate tensile capacity, Q_S, of a drilled and grouted pile in clay may be equal to the summation of the skin friction along the pile length. It can be expressed as:

$$Q_f = \P \int_{O}^{L} f_s \; D_{gs} \; dL \qquad (5\text{-}1)$$

where:

f_s	=	unit skin friction at grout-soil interface
dL	=	increment of insert pile length
D_{gs}	=	diameter of pile at the grout-soil interface
L	=	length of pile shaft

As discussed in Chapters 2.0 to 4.0, the unit skin friction, f_s, is affected by a number of factors including soil type, soil undrained shear strength, penetration depth below mudline, construction methods, time following installation, shaft displacement, and loading characteristics. Usually, the unit skin friction is estimated using either empirical correlations, total stress methods, or effective stress methods.

<u>**Empirical Correlations.**</u> The methods essentially relate the results of in situ tests such as the standard penetration test (SPT) and the cone penetration test (CPT) to the developed unit skin friction. They are used when no other data are available and should only be applied for preliminary design purposes.

CPT results and skin friction correlations are available for driven piles (de Ruiter et al., 1979). For drilled and grouted piles, it is possible to correlate CPT data to basic soil properties (such as shear strength) to indirectly predict unit skin friction using total or effective stress method.

<u>**Total Stress Methods.**</u> There are several methods to determine skin friction for clays. They range from setting the skin friction as a function of the undrained shear strength, S_u (API, 1986a), as a function of the soil $c/\bar{p}$ ratio (API, 1987; and Randolph and Murphy, 1985) or as a function of both (Semple and Rigden, 1984). These methods are based on driven pile case histories and cannot be directly applied for drilled and grouted pile design.

For drilled and grouted piles, the following equation is often used:

$$f_s = \alpha S_u \tag{5-2}$$

where: α is an empirical load transfer factor.

The α value used is normally determined from correlations between pile-load test results and undrained shear strength (Kraft and Lyons, 1974; and Reese and Wright, 1977), as shown in Figure 5-5. The large scatter in α values for drilled and grouted piles, as tabulated in Figure 5-6, indicates that considerable judgement and experience are required to choose a suitable α value. The following factors contribute to the uncertainty of this method when applying to offshore drilled and grouted piles:

1) The α values are based on a limited number of pile load tests on relatively short, stiff bored piles or drilled shafts.

2) No stress history is taken into account.

3) Offshore sampling and testing does not yield undrained shear strengths necessarily consistent with those obtained onshore and which were used to develop the empirical correlations.

4) Even though some efforts have been made to account for the effects of onshore installation methods on the α values (Reese et al., 1977), relating these to quality control offshore will be difficult.

Considering uncertainties discussed above, a full scale pile load test at a proposed TLP site would be invaluable for optimizing the design of drilled and grouted pile for an important structure such as a TLP.

Effective Stress Methods. Several methods for driven piles exists and some are still in the development stages. They include (1) the λ - method (Vijayvergiya et al., 1972; Vijayvergiya, 1977) and modified λ - method (Kraft et al., 1981b), (2) the β - method (Chandler, 1968; Burland, 1973; Meyerhof, 1976; Parry and Swain, 1977; Vesic, 1975; and Flaate and Selnes, 1978), and (3) general effective stress method (Esrig et al., 1979).

For drilled and grouted piles, there is still no established method. However, Burland (1973) has proposed using his β method for bored piles in heavily overconsolidated London clay in accordance with the following equation:

$$f_s = \beta \bar{p} \tag{5-3}$$

where: $\bar{p}$ = effective overburden pressure

β = dimensionless coefficient generally taken as 0.8 for London clay

Selecting values of the β coefficient in other clay deposits will require a great deal of engineering judgement.

5.4.2 Ultimate Axial Capacity in Cohesionless Soil

Because the use of drilled and grouted piles in cohesionless soils is uncommon, very few load tests are available to calibrate predictive methods. The conventional approach for driven piles (API, 1986a) is normally used. However, Ehlers and Ulrich (1977) have indicated that this approach is too conservative for long, slender piles. They recommend the following equations:

$$f_s = 0.7\, P_g \tan \delta \tag{5-4}$$

where: P_g = effective grout pressure (total grout pressure minus formation hydrostatic pressure)

δ = friction angle between sand and grout

However, the authors indicated that the pile-grout bond strength must be considered in pile capacity estimation because limiting values of skin friction are not applied with this method.

Touma and Reese (1974) recommended the estimation of f_s as

$$f_s = \beta_{ave}\, \bar{p} \tan \delta \tag{5-5}$$

where: β_{ave} = average value of load transfer ratio

$\bar{p}$ = effective overburden pressure

δ = friction angle between sand and grout

The authors recommended a β_{ave} of 0.7 for piles shorter than 25 ft. Values of β_{ave} can be less for longer piles. These values are based on five compression load tests of 24 to 36-in. diameter piles and may not be valid for long piles and for other sites.

There are many other design recommendations developed for bored piles and drilled shafts based on limited field and lab test data. These methods are generally presented in one of the following forms:

1) $f_s = K\bar{p} \tan \delta$ (5-6)

2) $f_s = a \, Pg \tan \delta$ (5-7)

3) $f_s = b_1 N + b_2$ (5-8)

4) $f_s = \beta \bar{p}$ (5-9)

5) $f_s = \beta \bar{p} \sqrt{\dfrac{v_p + v}{v_p - v_o}}$ (5-10)

where:

K = coefficient of earth pressure at rest

a = dimensionless coefficient ($a = 0.7$ as recommended by Kraft and Lyons, 1974)

N = standard penetration resistance blow count

b_1, b_2 = dimensionless coefficients

β = dimensionless coefficient = $(1 - \sin \phi) \tan \delta$

v_p = volume of the pile

v_o = volume of primary grout

v = total volume of grout

$\bar{p}$ = effective overburden pressure

Pg = grout pressure

δ = interface friction angle between soil and grout, which may be equal to ϕ when failure occurs within the soil itself.

These design methods were compared with the results of load tests on drilled piers in gravel and sand (Farr and Aurora, 1981). It was found that the frictional capacity of the piers were significantly higher than those predicted.

There is very little or no information available for the design of piles in sandy silt. It is expected that such soils would exhibit frictional behavior similar to those of sands thus the same approach would likely apply but this needs to be substantiated for offshore foundations.

5.4.3 Ultimate Axial Capacity in Special Soils

Pile load tests in soils other than clays and sands have been performed mostly on driven piles. Special soils, whose frictional behavior and axial load carrying capacity have recently been investigated, include the following:

Calcareous Soils. These soils are typically encountered in the temperate offshore zones such as Australia, Brazil, Florida and India. There are several publications on (1) the general properties of calcareous sands (Demars and Chaney, 1982), (2) their behavior during onshore driving of steel H-piles (Dougherty and Hunt, 1982) and steel pipe piles (Angemeer et al., 1975; Dutt and Cheng, 1984; Dutt et al., 1985; and Nauroy and Le Tirant, 1985), (3) instrumented model pile tests in calcareous sands under static and cyclic load (Lu, 1986), and (4) load tests on drilled and grouted piles in calcareous deposits (Angemeer et al., 1975; Nauroy and Le Tirant, 1985).

From the load tests and laboratory studies mentioned above, it has been generally concluded that the ultimate capacity of drilled and grouted piles is greater than that of driven piles. This was reasoned to be due to grain crushing caused by pile driving resulting in a reduction in lateral stress. Lu (1986) indicated that the ultimate capacity of piles in calcareous sediments depends on a number of interrelated factors and that grain crushing alone cannot fully explain observed pile behavior in calcareous sediments.

The greater frictional capacity of drilled and grouted piles was also reasoned to be due to permeation of grout into the sand formation, thus enlarging the failure surface.

Datta et al (1980) stated that, for uncemented calcareous sand, current methods of estimating f_s are too conservative in non-crushing sand. On the other hand, the methods are considered unsafe for sands exhibiting a great deal of crushing.

Weak Rock or Chalk. The subject of axial capacity of piles in weak rock was addressed in a conference on the subject (Institution of Civil Engineers, 1977). A number of load test programs have been performed (Searle and Bartholomew, 1977; Hobbs and Robins, 1977; Lord, 1977; Mallard and Ballantyne, 1977; Fragio et al., 1985; Settgast, 1980) in an attempt to better understand the uplift capacity of piles in chalk. Both tension and compression tests were performed to determine skin resistance on driven piles, cast-in situ piles, and Franki piles with enlarged bases. Chalk classification is often based on the SPT blow count, N, after Hobbs (1977), and is thus very subjective.

The results of the various testing programs can be summarized as follows:

1) Bored, cast-in-place piles have considerably more frictional capacity than do driven piles. Pile driving tends to destroy chalk structure, resulting in the formation of a "chalk slurry" around the pile. Resistance due to this slurry is not as great as that which can be obtained by concrete-to-intact chalk contact.

2) Cast-in situ piles exhibit increasing skin friction capacities with depth.

3) Chalk strength significantly affects the frictional resistance of bored piles, since the concrete is intimately associated with the intact chalk.

4) For short (16 ft) bored piles in mudstone, values of α were as low as 0.2.

5) The range of observed values of frictional resistance given in the literature shows considerable scatter. Suggested conservative values for maximum frictional resistance of piles in chalk are 0.72 ksf in weak chalk and 2.25 ksf in medium hard chalk.

6) Scale effects for tension capacity of piles in chalk are probably considerable. An order of magnitude difference in f_s is reported to exist between medium size piles (1 to 2 ft diameter) and smaller anchor piles. The existence of fissures in the chalk mass is clearly a major contributing factor.

7) Frictional characteristics of driven piles and drilled and grouted inserts tested in carbonate rocks (Settgast et al., 1980) reveal significantly higher values of bonding for the grouted insert (11 ksf) versus the driven piles (3-4 ksf).

8) The results of drilled and grouted pile load tests in calcareous claystone (Fragio et al., 1985) revealed a strain softening effect on t-z curves at depth. Simple elastic-perfectly plastic t-z curves were found to approximate the load-displacement behavior well.

In conclusion the literature contains no rational design method for piles in weak rock or chalk, and the results discussed above are those of pile load tests. All researchers indicate that further work is necessary before an attempt to predict the uplift capacity from strength tests alone can be safely accomplished.

Silts. Specific information on the axial capacity of piles in silts is scarce. Generally, the behavior of piles in silts is modelled as for a pile in cohesionless material. This may not be necessarily appropriate and each individual case should be evaluated on the basis of clay-silt-sand content, permeability, plasticity, and measurable undrained shear strength. After full evaluation of whether the silt behaves as a cohesive or cohesionless material, it may be necessary to analyze the deposit under both circumstances.

5.4.4 Group Effects

Piles that are closely spaced and rigidly connected together generally act as a group. This may change pile behavior (capacity, load transfer, settlement) as compared to that of single piles. Available test data for full-size pile groups are scarce and limited to compressive loading. No information seems to exist for drilled and grouted piles.

Procedures derived for driven piles may be used for drilled and grouted piles. Such procedures are based on group efficiency factors which represent the ratio of the group capacity to the summation of all the individual capacities. The group efficiency factor depends on size and shape of the group, spacing, relative length of the piles, construction procedures, and soil type. There are a number of empirical efficiency factors proposed and used but no widely accepted theory exists at present.

There have been several studies of pile group efficiency in clay. These are based on square group models (Whitaker, 1957; Barden and Monchkton, 1970; Sowers et al., 1961; Saffery and Tate, 1961; O'Neill, 1983) and circular group models (Matlock et al., 1982a). Based on the results of medium-scale load tests in clay (O'Neill, 1983), efficiencies of about unity for pile groups of average width to diameter ratios of 2.5 to 4.0 have been recommended.

For a width to diameter ratio of 2.0, the general trend is to decrease efficiencies to 0.5. It is also a common practice to conservatively use a group efficiency of 2/3 for driven and bored piles for typical pile spacings of 2.5 to 4 (Poulos and Davis, 1980; Reese and Wright, 1977).

Group efficiency as a function of pile spacing, size of pile, and size of group is shown in Figure 5-7 (Vesic, 1975) for both clays and sands. Groups of large piles seem to have efficiency factors closer to unity than groups of small piles. This result, however, is not totally conclusive in view of the limited number of load test data available on large piles. Also, efficiency factors found from small scale model tests in sands could be high and may not be safe to apply to groups of large piles as in offshore applications. For clays, the results from small model tests indicate efficiency factors much less than unity which could lead to conservative designs since a limited number of large-scale test results (AREA, 1951; Schlitt, 1951) show that the efficiency approaches unity. In practice, the values suggested by Vesic (1974) and Meyerhof (1976) combined with those from local experience would provide a reasonable range of efficiency factors used for design.

The load-transfer and load-deformation of piles in a group are also different from those of single piles. The group effect is expressed in terms of a deflection factor, which is defined as the ratio of the pile-group displacement to the displacement of a single pile carrying the same amount of average load per pile. Group displacements can be estimated by methods ranging from empirical approaches (Skempton et al., 1953; Meyerhof, 1959; Vesic, 1967; Mansur and Hunter, 1970) to quasi-theoretical methods (Poulos, 1971 and 1980; Focht and Koch, 1973; O'Neill et al., 1977). The latter are based on either elastic solid or shear transfer function approaches.

Existing quasi-theoretical methods usually over-estimate the group deflections (Matlock and Lam, 1980). Using group p-y (Matlock et al., 1982a) and t-z curves (O'Neill et al., 1982) generated from available test data would be a more realistic approach.

5.4.5 Cyclic Loading Effects

<u>Pile in Clays and Sands</u>. Results of load tests on full scale (Sangrey, 1977) and model (Holmquist and Matlock, 1976) driven piles show that two-way cyclic loading reduces the pile capacity to between 80 to 100 percent and 30 percent of the static capacity, respectively. Recent studies, as listed in Figure 5-8, show similar trends. The researchers found that the strain softening effect on the load-deflection curve is most pronounced for the initial loading after consolidation. As the number of cycles increases, or the consolidation time decreases, the softening effect decreases.

No relationship can be found between pore pressure variation and frictional resistance, (Bogard and Matlock, 1979; Holmquist and Matlock, 1976; Karlsrud and Haugen, 1983; Grosch and Reese, 1980). Thus it is likely that the degradation is mainly related to changes in the clay structure along the shear zone.

Based on model test data, Doyle and Pelletier (1985) reported that cyclic degradation is insignificant for one-way cyclic loading where shear stress reversal does not occur. This conclusion should be used with caution, since large one-way cyclic loads, especially on long piles, can produce shear reversal along the pile-soil interface and result in a significant degradation (Bogard and Matlock, 1979; Matlock and Lam, 1980). For two-way cyclic load, significant degradation (as much as two-thirds of the initial peak capacity) has been observed. Felio (1985) and Briaud and Felio (1986) found that there is a threshold level above which degradation starts to occur. These threshold values, which are summarized in Figure 5-8, can be related to the ratio of the residual friction over the peak value in a static loading test. Felio (1985) also reported that degradation is more severe in laboratory soil sample tests than in pile model tests. This would indicate that it is incorrect to represent cyclic behavior of piles in clay by using results directly from laboratory soil tests. This suggestion is contradictory to procedures presented by Karlsrud and Haugen (1983), however.

The above discussions are based on cyclic degradation behavior on a t-z curve basis. Appropriate loading patterns, elastic properties, and soil resistance characteristics should be incorporated into an analytical model to evaluate the cyclic degradation behavior on a total pile basis. A general framework of analysis incorporating a cyclic degradation algorithm should provide a good basis for such analyses. Correlation with the pile load test data for a wide variety of loading patterns is needed to improve the present state of the art.

Piles in Calcareous Soils. Angemeer et al (1975) reported that cyclic loading of grouted piles in calcareous sediments produces an insignificant loss in frictional capacity after about 90 cycles of variable wave loading. This seems to be contradictory to other work by King et al (1980) on small-scale pile segment tests in situ, which showed that large cyclic displacements can significantly reduce frictional resistance.

Lu (1986) performed cyclic load tests on model driven piles in calcareous sands and concluded the following:

1) Shear reversal produces significant degradation in frictional resistance of the pile.

2) Cyclic loading produces significant degradation in pile stiffness and frictional resistance and the degradation can occur even at a small magnitude of cyclic pile displacement.

3) The design of grouted piles in calcareous sand to resist cyclic loading should place emphasis on allowable displacement rather than the conventional pseudo-static "factor of safety" approach.

It is anticipated that the cyclic load behavior of drilled and grouted piles, although quantitatively different, will be similar to that of driven piles in calcareous sand. Thus, the above conclusions by Lu (1986) should be useful in designing drilled and grouted piles.

5.4.6 Rate of Loading Effects

Environmental loads acting on offshore platforms are typically applied at faster rates than loads applied to test piles. This fast loading rate results in not only the inertia effect from the pile mass, but also an increase in the load-carrying capacity of the soils. Casagrande and Wilson (1951) and Taylor and Whitman (1953) reported a soil strength increase of as much as 100 percent over the conventional static strength. The increase is much lower for sands. Sangrey (1977) also indicated an increase of 10 to 20 percent in strength for marine soils under short term loading. A log-linear relationship between loading rate and measured strength is shown in Figure 5-9.

Rate effects on the axial and lateral response of piles was initially reported by Bea and Audibert (1979) and Audibert and Dover (1982). These authors concluded that rate of loading effects could range from 5 to 15% per log cycle change in rate of loading. Rate effect on vertical and lateral pile response was further studied by Briaud and Terry (1986), who proposed the following model:

$$\frac{Q_{u1}}{Q_{u2}} = \left[\frac{t_2}{t_1}\right]^n$$

where:

Q_{u1} = ultimate pile capacity for loading period t_1
Q_{u2} = ultimate pile capacity for loading period t_2
t_1 = loading period t_1
t_2 = loading period t_2
n = viscous exponent depending on clay type

The viscous exponent n is a material specific and site specific parameter which varies with S_u, plasticity (PI), liquidity index (LI) and overconsolidation ratio (OCR). Briaud and Terry (1986) reported that n can be estimated from the following:

$$n = 0.44 \left[\frac{S_u\ (ref)}{P_a}\right]^{-0.22}$$

$$n = 0.028 + 0.00060\ w\ (\%)$$
$$n = 0.035 + 0.00066\ PI\ (\%)$$
$$n = 0.036 + 0.046\ LI$$

where:

S_u ref = reference undrained shear strength at failure time of 1 hr.
P_a = atmospheric pressure
w = moisture content

Sufficient full scale fully instrumented pile load test data will be needed to verify this correlation. Consideration of rate of loading effects could be useful in the design of offshore piles under storm loading. The ultimate capacity would be increased, resulting in a substantial reduction in pile penetration required.

5.4.7 Load-Deformation Approach.

The axial pile capacity methods discussed previously are of the limit equilibrium type, where the skin friction is assumed to be simultaneously fully mobilized throughout the pile length. Shear-deformation (t-z) behavior at the soil-pile interface is not taken into account. These methods are adequate for short, stiff piles, as often used onshore. For long, flexible offshore piles, axial deformations differ along the length of the pile, resulting in different levels of mobilized skin friction. Some efforts to incorporate the load and deformation behavior of the soil-pile system to predict the pile performance are discussed below:

<u>**Elastic Solid Approach**</u>. In this approach, the soil and piles are modelled as a continuum. Displacements in the soil are calculated using either elastic theory (Mindlin, 1936) or the finite element method. The fundamental limitation of this approach is that the effects of pile installation (residual stresses, compaction, etc.) are not included and large shear displacements are not incorporated in the soil model. Therefore, the shear transfer function approach described below is preferred.

<u>**Shear Transfer Function Approach**</u>. In this approach, discrete elements are normally used whereby the pile is modelled as an elastic rod and the soil support by a series of linear or nonlinear springs along the pile shaft (t-z curves) and at the pile tip (q-z curves). The pre-determined t-z and q-z relationships are input in various computer programs which incorporate the above models. These programs include AXCOL (Matlock et al., 1981), DRIVE (Matlock and Foo, 1979), DANA (Gates et al., 1977) and INTRA (Arnold et al., 1977).

Figure 5-10 depicts the discrete element model used in the computer program DRIVE. In this program, loading functions can be generated to simulate different load conditions. The program can therefore be used to investigate the stability of piles under cyclic tension loading. The soil supports can be linear, nonlinear and hysteretic with degradation parameters, and can be either empirical or calculated from the analysis of a single-slice using the CASH program.

The CASH program models the soil reaction against a short segment along the pile as a function of the time-history of the displacement of the segment (Fig. 5-11). It includes the effects of cavity expansion, pore water migration and soil consolidation. The result can be used as an input to the DRIVE program. It should be noted that these programs are only tools for rational analysis. They can only be as good as the input parameters. All judgments and interpretations are left to the engineers who use them.

One of the most important input parameters for the programs discussed above is the shear transfer functions or shear transfer-displacement (t-z) curves. A number of publications have presented methods to develop t-z curves for both driven and drilled and grouted piles (Seed and Reese, 1955; Kezdi, 1957; Reese, 1964, Coyle and Reese, 1966; Coyle and Sulaiman, 1967; Reese et al., 1969; Holloway et al., 1975; Kraft et al., 1981a; and Kraft et al., 1981b).

Reese et al (1969) presented a closed form t-z relationship for bored piles based on load test data of a 30-in. OD, 28 ft penetration bored pile. Kraft et al (1981b) presented a more recent t-z development method which also provides for strain softening of the soil after peak resistance. Results from a series of load tests on drilled and grouted piles in clay (Engeling and Reese, 1974) indicates that t-z curves for drilled and grouted piles are similar to those suggested for driven piles.

A number of researchers (Coyle and Reese, 1966; Vijayvergiya, 1977; Holmquist and Matlock, 1976) recommended various t-z curve shapes. Those recommended by Holmquist and Matlock represented the most comprehensive approach for clay and conformed very well to the strain softening behavior and the reduction in capacity reported by Murff (1980) for long piles.

In summary, for piles in clay, methods recommended by Coyle and Reese (1966) or by Holmquist and Matlock (1976) are commonly used. For piles in sand, the method suggested by Vijayvergiya (1977) is usually followed. These t-z curve construction methods all suffer from a limited data base of pile load tests. This represents the major limitation of the shear transfer function approach until more full scale, fully instrumented pile load tests are available to check further their validity.

5.5 Lateral Behavior

5.5.1 General

Although the lateral load component for TLP foundations is expected to be only about 10% of the axial component, it significantly affects the configuration of the top portion of the pile foundation. In addition, in earthquake-prone areas, seismically-induced lateral loading may be a significant factor in pile foundation design.

Specific methods to evaluate lateral load behavior of drilled and grouted piles do not exist even though some are available for bored piles (Reese and Welch, 1975; Reese and Allen, 1977).

5.5.2 Subgrade Reaction Method

Many investigators have put their effort on the determination of a coefficient of horizontal subgrade reaction (Broms, 1964a, 1964b, 1965; Kubo, 1968; Davidson and Prakash, 1963). After values of subgrade reaction at each depth are selected, the deflection and curvature of the pile is calculated using methods based on the theory of beam on elastic foundation.

One major pitfall of the methods mentioned above is that they generally ignore the nonlinear nature of the subgrade reaction. Instead of calculating the deflection at the design load level, the methods give an ultimate load to which the designer applies a factor of safety. A higher safety factor may be used in an attempt to account for cyclic load effects.

It is a common practice not to use design loads exceeding one-half the ultimate resistance to ensure that the soil-pile system remains in its elastic range, even though the pile may in fact undergo inelastic deformations.

5.5.3 P-y Method

The load-deformation method is often referred to as the p-y method. The pile is modeled as an elastic beam or column and the soil as a series of nonlinear springs. Different computer programs such as BMCOL 76 (Matlock et al., 1981) and COM 622 (Reese, 1977) have been developed to analyze pile behavior under static or quasi-static lateral loads. Some programs such as SPASM (Matlock et al., 1978), DANAS (Gates et al., 1977) and INTRA (Arnold et al., 1977) are available to analyze cyclic loading conditions. DANAS and INTRA can also model the platform structure for soil-structure interaction analysis. These methods require an input of lateral soil resistance-deflection (p-y) curves, which generally are nonlinear.

The methods outlined in API RP 2A (1986a) are usually adopted to construct p-y curves. These methods stem from the works presented by (1) Matlock (1970) for soft clay; (2) Reese et al (1975) for stiff clay; and (3) Reese et al (1974) for sand. These methods are based on a limited number of field and laboratory model pile tests together with laboratory soil strength tests.

The p-y curve construction procedures as proposed by Matlock (1970) are shown in Figure 5-12. The procedures account for the cyclic load behavior of the soil-pile system which exhibits degraded hysteresis with gapping or plastic flow at shallow depths.

Because most of the test data are limited to piles of small diameter, the above p-y procedure may need to be modified to account for the increase in p-y resistance for larger diameter piles (Stevens and Audibert, 1979). Two recent studies (Matlock and Cheang, 1986; and Lam and Martin, 1986) suggested that the increase in soil resistance maybe due to the additional moment resistance caused by axial resistance to pile rotation, which is not considered in the conventional p-y methods. This increase is more pronounced for large diameter piles with large pile head rotation (i.e., free head condition), and for overconsolidated stiff clay. For normally consolidated clays, the effect of rotational resistance is negligible for pile diameters up to 5 feet.

For cohesionless soils such as sands and silts, p-y procedures adapted from Reese et al (1974) and outlined in API RP 2A (1986a) are commonly used. They are based on small diameter pile tests in sand and incorporate a strain-hardening soil resistance. Reduction factors are included to account for cyclic loading effects. Information on the material's relative density are required to model the initial modulus of the p-y curve.

A disturbing feature of the above p-y curve formulations is that the deflection is a direct linear function of the pile diameter for all soils (note that the coefficient of subgrade reaction is an inverse function of the pile diameter). The methods are based on limited number of pile load tests which were performed on piles of only one diameter at each location. Thus linear relationships between deflection and pile diameter are not experimentally well established and should be used with caution particularly for piles with diameters drastically different from those used to develop the above p-y method.

It is possible that the deflection at ultimate lateral resistance is a function of diameter until plane strain conditions start to dominate. However, this transition point cannot be defined at this time. Vesic (1961) showed that for an infinitely long strip on elastic subgrade, the coefficient of subgrade reaction is a function of the width to the 1/3 power. Experiments or rectangular plates up to six inches wide (Vesic and Johnson, 1963) generally agree with these results. Lateral load tests on large diameter piles are needed to further clarify this issue.

5.5.4 Other P-y Methods

Other methods have been proposed to construct p-y curves from experimental data. Menard (1962) developed a p-y curve formulation based on a subgrade modulus measured with the Menard pressuremeter, as shown in Figure 5-13.

Baguelin and Jezequel (1972) performed lateral load tests on piles embedded in submerged loose to firm silt and fine sands. Their experimental p-y curves compared well to the load deformation curves obtained using their newly developed self-boring pressuremeter. Their results were also compared to the p-y formulation by Menard and showed that the Menard method tended to overpredict the pile deflection significantly.

5.5.5 Special Soils and Rocks

Specific methods to determine the lateral resistance of soils other than clays and sands are generally not available. Soils such as silts, calcareous sands, hydraulic fill and dredged material are usually generalized into either cohesive or cohesionless type and the corresponding p-y methods are then applied. In some cases, site specific procedures are developed from the results of a pile load test or from hindcasting analytical studies.

Hagenaar et al (1986) performed lateral load tests on drilled and grouted piles in carbonate rock and soils. They applied the p-y method for stiff clay (Reese et al., 1975) to model the strain-softening behavior of the carbonate rock and used a beam-column type analysis. They reported that the predicted displacements agreed reasonably well with the measured data.

Fragio et al (1985) also performed lateral load tests on steel pipe piles drilled and grouted into calcareous claystone. They used modified p-y curves to model the claystone as an elasto-plastic material below a critical depth above which surface effects caused strain softening. They reported fairly good agreement with the load test results.

5.5.6 Group Effects

Focht and Koch (1973) combined the p-y methods by Matlock and Reese et al, with the elastic method by Poulos (1971), resulting in modified p-y curves which give a group lateral load capacity less than the sum of the individual capacities for pile spacing less than about five diameters. This method, however, has no published calibration.

Kim and Brumgraber (1976) presented the results of lateral load tests on individual and groups of driven piles in medium to stiff clays. They showed that the group capacity is about 1.5 to 2.0 times the sum of the individual capacities, which is in direct conflict with what would be predicted by the Focht and Koch (1973) method. There is some question about the validity of these results, as the pile cap was resting on the ground surface and may have contributed some additional load resistance. Therefore, reexamination of these results is necessary, and the methods to determine the group effect developed by Focht and Koch (1973) need more extensive calibration against well controlled experimental data. Further research on this subject is necessary to resolve the problem.

5.5.7 Cyclic Loading Effects

As was the case for cyclic axial loads, cyclic lateral loads cause degradation of lateral soil resistance. In addition, a gap or plastic flow may develop around the pile at shallow depth resulting in further reduction in lateral pile resistance. This phenomenon has been shown to occur in both clays and sands and is expected to occur in other soil types as well. While no specific recommendations could be found for drilled and grouted piles, it is believed that recommendations such as found in API RP 2A (1986a) for driven piles are equally applicable to drilled and grouted piles.

5.5.8 Rate of Loading Effect

As discussed previously for axially loaded piles, Bea and Audibert (1979) and Briaud et al (1986) have presented data to be used in the evaluation of loading rate effects on lateral load behavior. These data are based on the test results on driven, jacked and bored piles, and drilled shaft in clays and sands. The p-y curves are adjusted according to the relationship between the duration of structure loading and the soil test loading using a power law similar to that used for axial loading.

5.6 Analysis and Design Considerations

Because of insufficient well-documented tension load test data for offshore drilled and grouted piles, the test results from onshore drilled shafts, piers, bored piles and soil anchors represent an invaluable source of data in the study of drilled and grouted pile behavior under tensile load. The following technology gaps have been identified:

1) Installation procedures significantly affect the tensile load behavior of drilled and grouted piles. Most of the methods attempt to account for these effects through varying degrees of empiricism, judgement and experience.

2) Most of the existing data came from bored piles, drilled shafts and piers of shorter length as compared to the long offshore drilled and grouted piles.

3) Quantitative assessment of degradation of the frictional resistance of drilled and grouted piles is not available.

To enhance the confidence level in the design of drilled and grouted piles for TLP foundations, it will be necessary to observe and improve our understanding of the behavioral mechanisms at every stage of construction. These include:

1) state of stress in soil before drilling,
2) stress change due to drilling and pile installation,
3) soil-grout interface configuration in relation to the installation technique,
4) failure mechanism and interface behavior under different loading stages.

In addition to the technology gap mentioned above, there are several factors that need to be considered in the design and installation of drilled and grouted piles. They have been regrouped under design-related and installation-related concerns.

Design-Related Concerns. The following factors may result in an unsafe or uneconomical design:

1) Lack of information on the drilled and grouted pile geometry.

2) Lack of well-documented load test data on drilled and grouted piles particularly on long offshore piles under cyclic tension loading.

3) Uncertainties in the extrapolation procedures for test data from small piles to large piles.

4) Lack of understanding of the behavior of the composite cross-section of drilled and grouted piles.

Installation-Related Concerns. These factors were previously discussed in Chapter 4.0 and can have a significant effect on the pile capacity. They include:

1) hydraulic fracture
2) stress relief in soil due to drilling
3) soil disturbance
4) mudcake formation
5) quality control

Possible remedial measures for the above items were discussed in Section 3.6 and 4.8.

6.0 MAJOR TECHNOLOGY GAPS AND RECOMMENDED RESEARCH AND DEVELOPMENT

6.1 General

This chapter summarizes the major technology gaps which require attention regarding drilling, grouting, and designing drilled and grouted piles. These are digested from the detailed discussions presented in Sections 3.6, 4.8, and 5.6. Areas of further research and development which should be pursued are also pointed out.

6.2 Drilling Technology

6.2.1 Borehole Stability

Without properly installed surface casings, cave-ins or collapse of borehole walls could result in an undesirable motion of the foundation template as well as in difficulties in placing the insert a pile into the borehole. Understanding the time-dependent behavior of borehole wall stability, and the lateral earth pressures under different working conditions will be helpful in the construction planning.

6.2.2 Soil Erosion at Seafloor Due to Casing Installation

Erosion and crater formation at the seafloor will also cause instability of the borehole. Evaluation of soil types and proper installation techniques based on past experience is important. Installation methods may need to be modified to minimize the incidence of such problems.

6.2.3 Drilling Fluids

Improper drilling fluid properties can result in either borehole cave-in or hydraulic fracture. Formation of a mudcake on the borehole wall also degrades frictional capacity of drilled and grouted piles. Knowing the effect of type, density and viscosity of drilling fluids with regard to hydraulic fracture and mudcake formation would be important.

6.2.4 Hole Verticality and Mechanical Disturbance

Hole verticality is an important factor for long, tightly grouped piles and can affect the process of running an insert pile or grouting around a non-concentric pile. Both hole verticality and mechanical disturbance could be minimized by selection of appropriate drilling and quality control methods.

6.2.5 Underground Obstructions

These could result in at least a delay of the drilling operation or, at worst, abandonment of the borehole. A careful geological survey including previous drilling information available in the vicinity will help. Improved geophysical equipment and methods to detect underground obstructions should be developed.

6.2.6 Quality Control

A major effort should be expended in defining and eventually implementing a quality control program when drilling boreholes for construction of drilled and grouted piles. A manual describing tolerances of construction (drilling mud weights and viscosity, hole verticality, recommended drilling procedures and drilling assemblies) should be developed for operators and drilling contractors to follow.

The important area requiring research and development is quality control of the drilling operation. Field studies should be initiated to assess the ability to monitor the drilling of vertical holes using downhole inclinometers. A review and study of present equipment to caliper boreholes in soft or unstable formations without causing excessive damage to the borehole walls will be needed. An automatic system should be developed to constantly monitor the mud weight, viscosity and temperature to indicate changes to the drilling fluid during drilling.

6.3 Grouting Technology

6.3.1 Gravity (Primary) Grouting in Deep Water

The problems associated with this construction phase are similar to those encountered during drilling, i.e., excessive pressure may cause hydraulic fracture. Multiple stage grouting appears to be a promising solution to alleviate the problem.

6.3.2 Excessive Grout Loss

Excessive grout loss is caused by hydraulic fracture and permeation of grout into permeable soils. The hydraulic fracture phenomenon needs to be further studied along with the related parameters such as soil characteristics, grouting pressure and grout flow rate. A laboratory or small scale field program may be necessary.

The relationship between soil permeability and various factors in the grouting process (grout viscosity, weight, rate of flow, grouting pressure) should be investigated. Also, effectiveness of bridging materials used to avoid grout loss in permeable soils should be evaluated.

6.3.3 Mudcake Formation

Removal of the mudcake formed on the borehole wall is an important consideration in order to maximize the load transfer capability between the pile and the formation.

Research must be performed to understand the behavior of mudcake formation in all soil types, define the effect of mudcake formation on pile load-deformation behavior following primary and secondary grouting, and investigate the effectiveness of grout preflushes and mechanical scrapers in removing mudcake. Performance of laboratory or field load tests on model and full scale pile segments in drilled boreholes with mudcake intact and with the mudcake removed would improve the understanding of mudcake effect on load transfer behavior.

Decompression of the borehole after drilling may occur during preflushing of the borehole prior to grouting. Studies must be made of the effect of stress relief at the borehole wall on hole stability and pile load transfer behavior following final construction. Laboratory or field load tests may be performed on model piles constructed while allowing stress relief of the borehole wall and no stress relief. Quantification of the effect of borehole stress relief on load transfer should be the final product of such an effort.

6.3.4 High-Pressure Grouting

The technique of grout reinjection or high pressure grouting requires additional investigation. The use of an inflatable packer system would have to be modified for use in deep water to overcome the high hydrostatic stresses. Also the effect of secondary grouting on pile capacity would need to be further quantified. A set of field or laboratory load tests on model pile segments could be performed to measure the load-deformation behavior before and after secondary high pressure grouting.

6.3.5 Quality Control

Quality control during grouting is crucial to ensure proper load carrying capacity of the pile foundation. Development of a manual is deemed necessary to provide operators and contractors with consistent guidelines. The manual could be a cooperative effort between a number of industry participants to produce a complete document on quality control procedures.

The important area for research and development is to define the effect of grouting quality on load transfer behavior of drilled and grouted piles. A field program to simulate piles constructed with good and poor grouting control should be considered.

Load tests on each pile would give some indication as to the effect of grouting quality on pile behavior.

Research and development should be expended for the systems and downhole tools necessary to provide the quantitative information on grout volume, thickness and quality of grout bonds. A field test program on pile segments constructed under a controlled condition with known configurations and degrees of quality may be planned to calibrate these tools. This effort would require coordination and input from groups currently involved in similar work such as Schlumberger, Welex or other geophysical logging companies. Tests in manufactured casings onshore with good and poor quality grout jobs would provide a data base from a controlled experiment. The effort will also necessitate coordinating with offshore operators to test modified or newly developed tools in recently constructed well casings.

6.4 Analytical and Design Methods

6.4.1 Site Geology and Geophysics

The number of previous geologic investigations in deep water sites may be limited, hence, the engineers and designers may be working in frontier areas where no previous data is available. The use of deep tow geophysical equipment to map the seafloor and subsurface conditions do not provide exact positioning capability and may not provide the necessary resolution for engineering purposes.

An effort is warranted to collect and catalogue the geologic information available in the public domain pertinent to the deep water regions of the major offshore fields of the world. The work would serve as a reference point and be available at geology clearing houses and major libraries.

It will be necessary to investigate the presently available geophysical systems and evaluate each with regards to developing information for engineering study of the seafloor. The evaluation should reveal each tool's ability to resolve geologic structure and topography, such as presence of gas, sediment layering, faults, previous slide surfaces and seafloor obstructions, in relation to the engineering analysis at the site. A significant research and development effort may be needed to provide exact positioning capabilities of deep tow and mid-tow geophysical systems, which are economical to operate.

6.4.2 Soil Sampling and Testing

Soil sampling at shallow-water offshore sites has been successfully performed with several types of sampling devices, such as driven, push, and piston samplers. In deep water, previous experience has shown that samples taken with these devices had been badly disturbed by the expansion of compressed pore fluid, dissolved gas and sublimation of gas hydrates.

Therefore, the major research need to benefit soil sampling would be development of a pressurized sampler for use in deep water regions. Pressurized samplers, in spite of their original purpose as research tools, would be valuable to the industry since they can operate as efficiently as other in situ tools. Hence, a development effort to design, fabricate and test a new prototype pressure sampler for production sampling would be warranted.

As a direct consequence of obtaining pressurized samples, a self-contained apparatus for testing soil samples under pressure must be developed. The test chamber must be capable of performing consolidation as well as strength tests on samples extruded from or contained in the pressurized sampler. A remotely operated system where the technician operates from outside the pressurization chamber appears to be the only viable solution.

6.4.3 Axial Pile Behavior

As discussed in various sections of this report, data on the axial behavior of drilled and grouted piles is not as complete as is available for driven piles. Several studies have involved performing load tests on short, drilled and grouted piles installed onshore under near ideal construction conditions.

A significant research and development effort is necessary to develop a predictive model for the axial behavior of drilled and grouted piles. The research program should include laboratory model tests, and then field tests on both small-scale and full-scale piles. The test program should address such variables as soil type, depth effect, soil stress history, time effects, pile construction quality and degradation of capacity due to cyclic loading.

The laboratory experiments could consist of model piles, 2 to 4-in. in diameter grouted into samples of reconstituted clay and sand manufactured in tall bins. A full suite of tests could be performed at relatively low cost to provide valuable input to future field tests.

Utilizing data from the above laboratory tests, a set of small-scale field experiments should be conducted for verification. The piles would be nominally 8 to 12-in. grouted diameter. Two sites should be located, one consisting of predominantly sand and the other of clay.

Full-scale drilled and grouted pile tests should be performed both onshore for better controlled conditions and offshore for more realistic working conditions. The drilling and grouting processes, including quality control procedures, will need to be studied and evaluated together.

6.4.4 Lateral Pile Behavior

Better methods of soil sampling and testing of the soft surficial sediments should be investigated first so that parameters regarding lateral soil resistance are more reliably derived. A program similar to, and perhaps in conjunction with, the program outlined for studying axial pile behavior should be performed. For this work, it may be important to install model and small-scale driven piles as a control for the experiment. Forecasting and hindcasting of the driven pile behavior using conventional models may shed light on the results of the lateral load tests on the drilled and grouted piles.

6.4.5 Steel-Grout Composite Section Behavior

Laboratory Study. The objective of a laboratory study would be to define the stress-strain behavior or stiffness of a steel-grout composite section under cyclic loading. This would be performed by designing model test sections to be loaded in axial tension and compression to sufficient strain levels to cause local failure and cracking of the grout. The test sections would model the stiffness ratio of grout and steel in a typical full-scale offshore pile as follows:

$$\left[\frac{(AE)_{grout}}{(AE)_{steel}}\right]_{\text{Full-Scale}} = \left[\frac{(AE)_{grout}}{(AE)_{steel}}\right]_{\text{Model}}$$

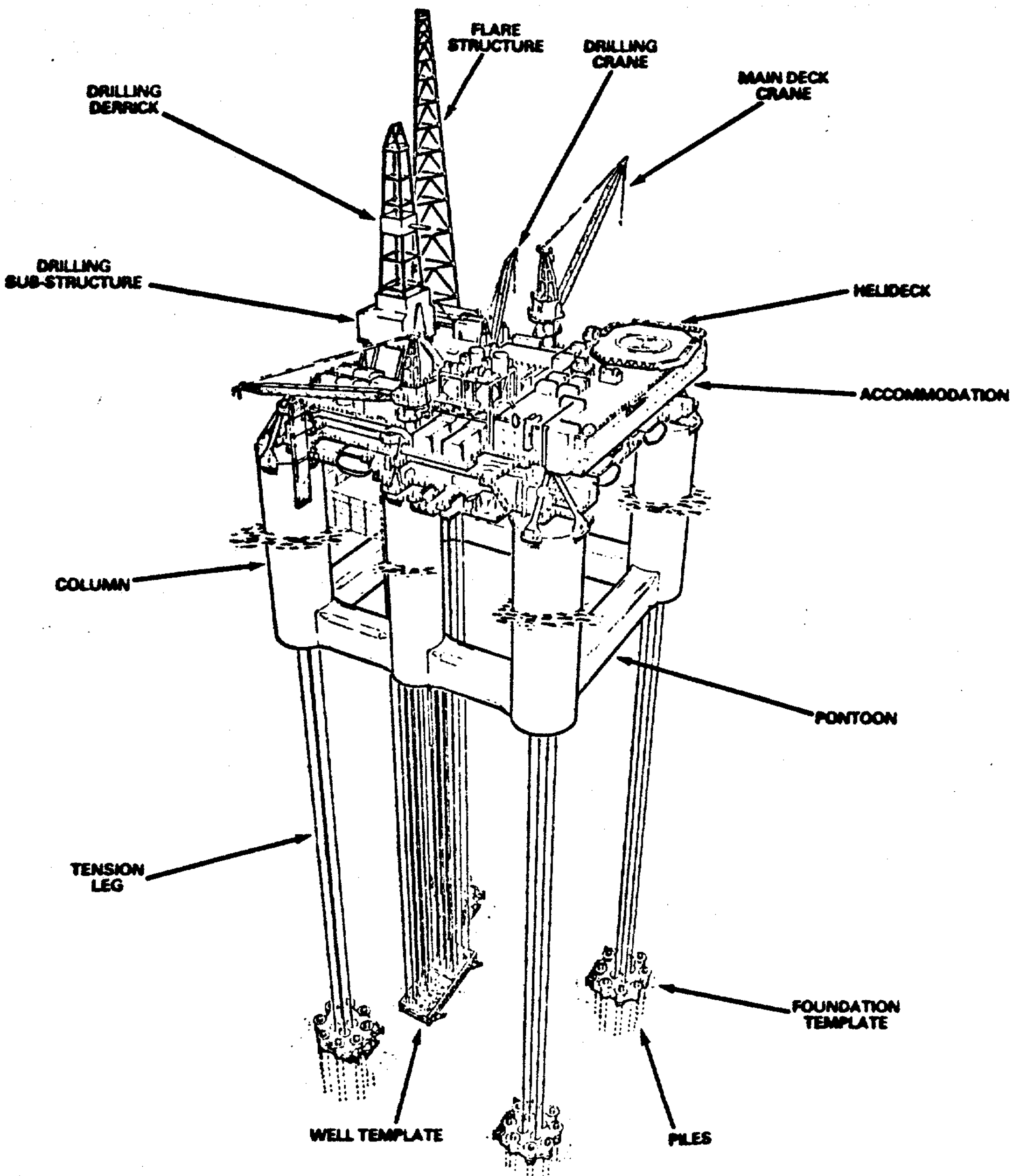
For example, the model pile section could be a 4-in. diameter schedule 40 pipe with a wall thickness of 0.28-in. encased in a grout skin 1.6-in. thick.

A set of preliminary load tests on an uninstrumented grouted section should be planned to quickly test the behavior of the composite section. Observation and documentation of crack propagation in the grout during loading would be performed.

A second set of tests could be performed on instrumented pipe sections. The pile sections would be tested to failure in axial tension under static loading conditions. Again, cracking of the grout and the corresponding strain levels would be noted.

A third set of tests would involve cyclic loading of the grouted test sections. The purpose of this set of tests would be to establish the effects of cyclic loading on the composite pile behavior. Emphasis would be placed on recording the degradation and stabilization of pile stiffness under low-to-near-failure cyclic stress levels.

Analytical Simulations. The objective of performing analytical studies would be to improve the present understanding of the behavior of a steel-grout composite section under axial loading through hindcasting of the previously described experiments. The scope of the effort would likely include performing a set of finite element solutions simulating various geometries of steel-grout composite sections subjected to various axial loading conditions (static, cyclic, tension and compression).


FLARE STRUCTURE
DRILLING CRANE
MAIN DECK CRANE
DRILLING DERRICK
DRILLING SUB-STRUCTURE
HELIDECK
ACCOMMODATION
COLUMN
PONTOON
TENSION LEG
FOUNDATION TEMPLATE
WELL TEMPLATE
PILES

HUTTON TLP

JOB NO.

OFFSHORE DRILLED AND GROUTED PILES: SUMMARY COMPARISON OF NORTH SEA EXPERIENCE WITH TLP REQUIREMENTS FOR DEEP WATER

Relevant Features	Thistle A and Piper Platform	Remarks
Installation time	Thistle A: Drill and grout: 9 piles in 53 days (6 d/p*) 25 piles in 81 days (3 d/p*) Piper: Drill and grout: 12 piles in 120 days (10 d/p*) Idle: 5 1/2 months Total time: 12 1/2 months Planned time: 2 months	For increased water depths and installation from a floating vessel, installation time could increase substantially without good planning.
Problems	- Cave-ins due to insufficient head from mud - Loss of circulation during drilling; sometimes at driven pile tip (probably due to loss of fluid in sand layer) - Holes squeezed inward which made insert pile installation difficult - Gumming of drill bit - Excessive grout take, typically 60% more than planned and in one case, 3 times that planned with grout found in adjacent hole - Equipment breakdowns, poor vessel support	Without remedial measures and development work, significant problems can be expected.

*d/p = days/pile

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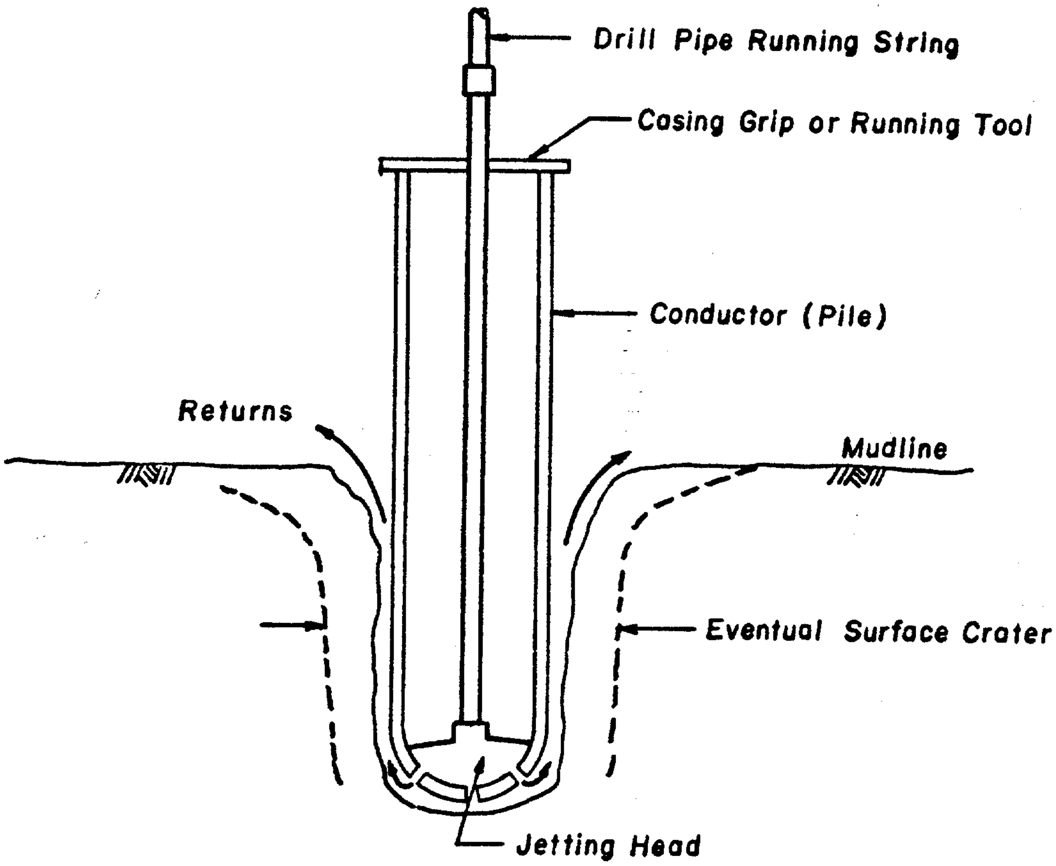
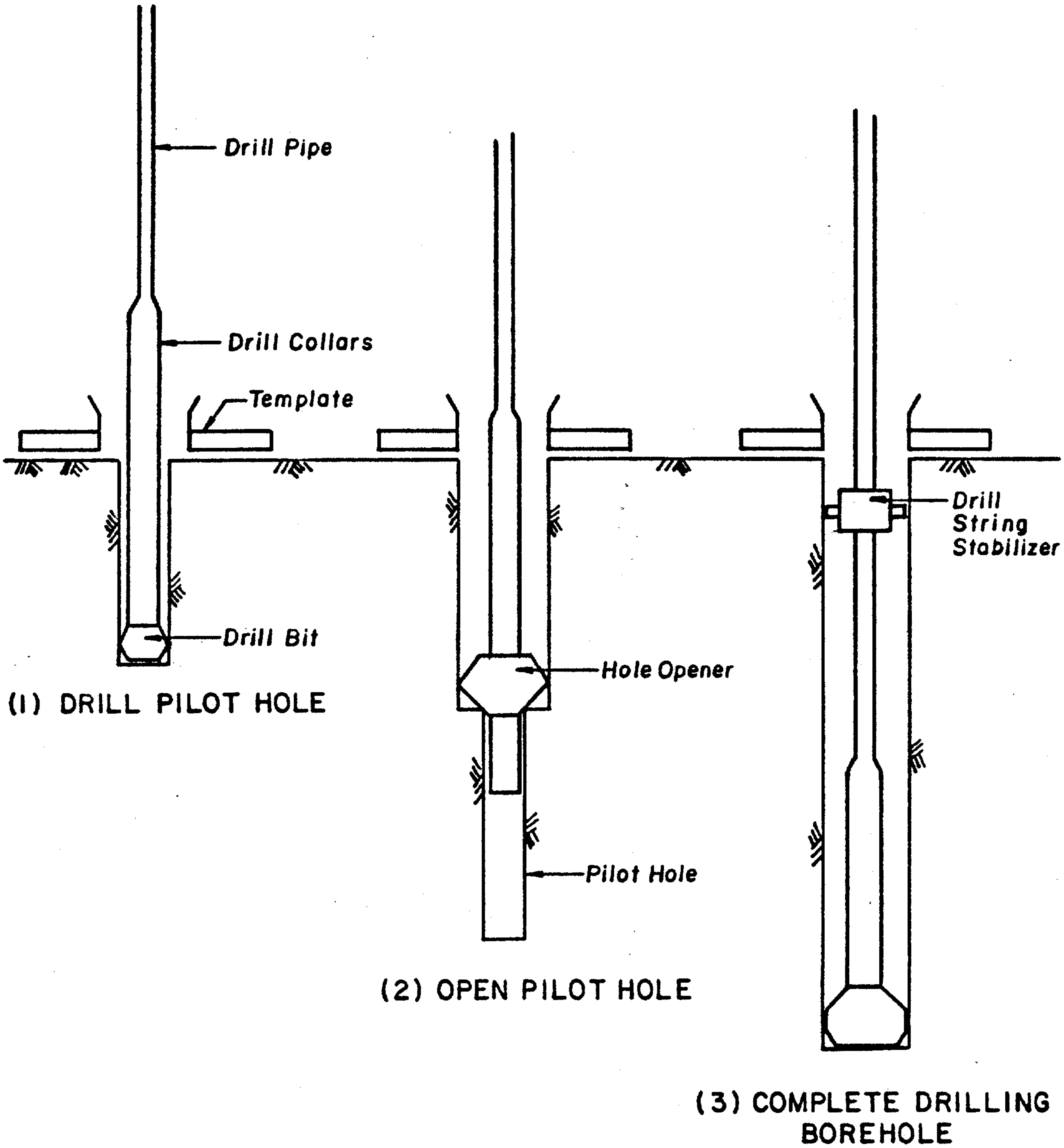
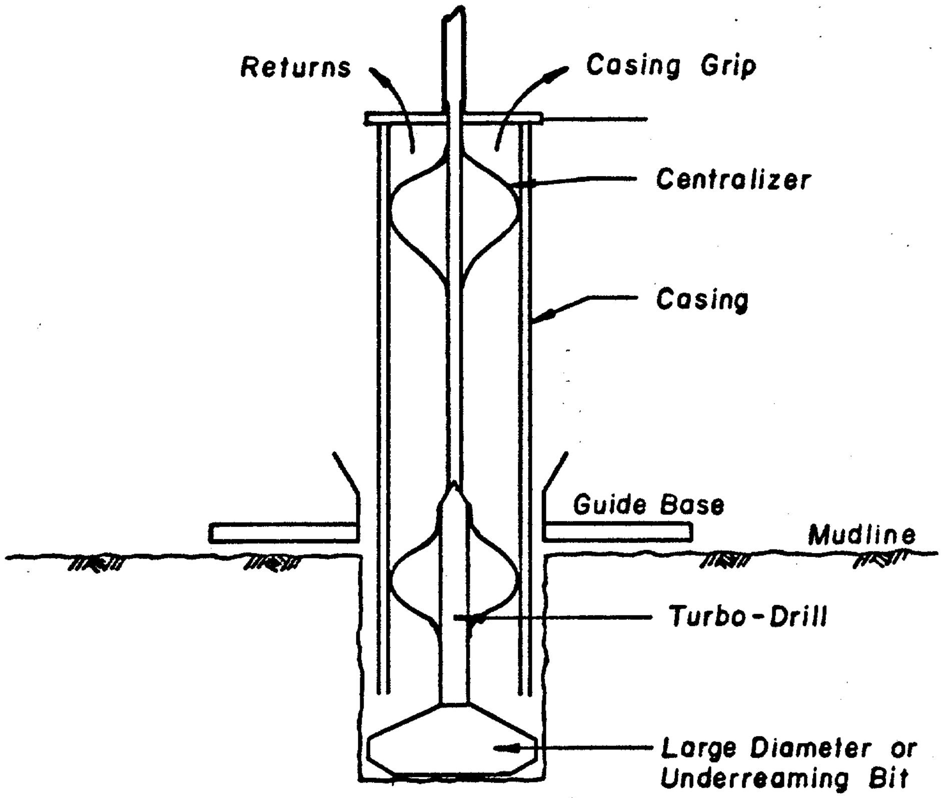
FIG. 2-1



FIXED JETTING ASSEMBLY

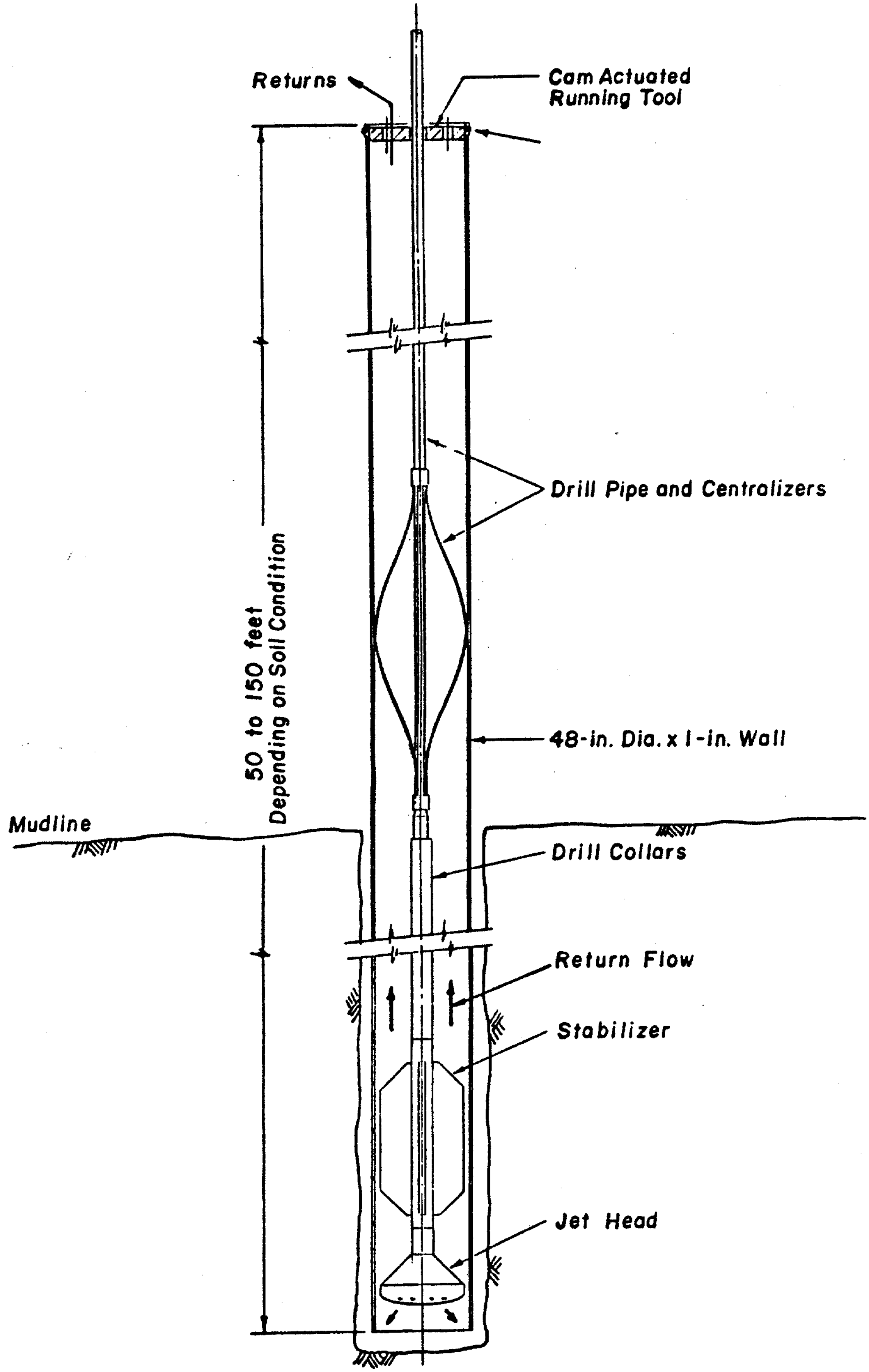


DRILLED-IN CONDUCTOR
USING A PILOT HOLE



NOTE: Keep drilling Bit just larger than Casing so that drilling fluid returns almost all inside Casing.

DRILLED-IN CONDUCTOR USING A TURBODRILL



JETTING WITH INTERNAL RETURNS

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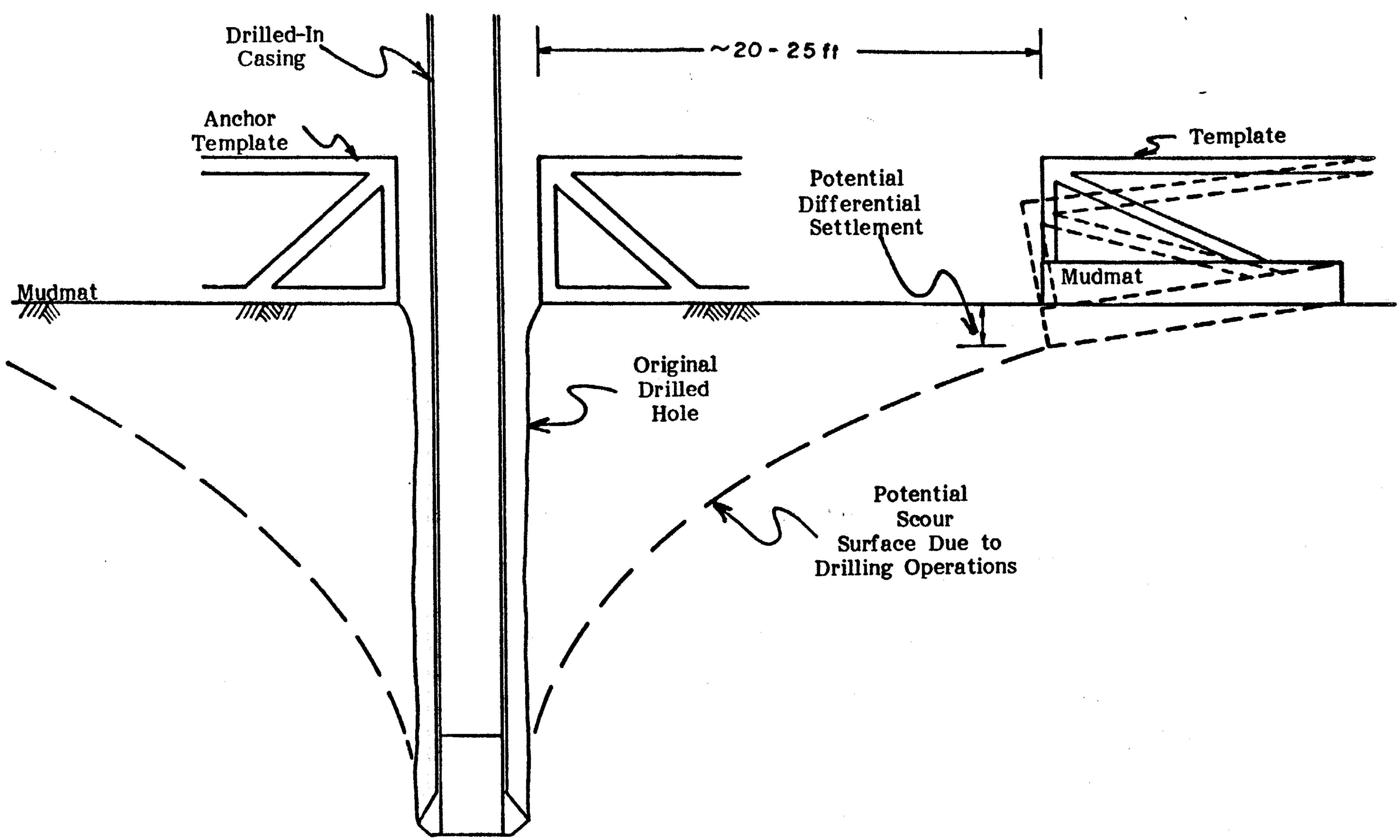
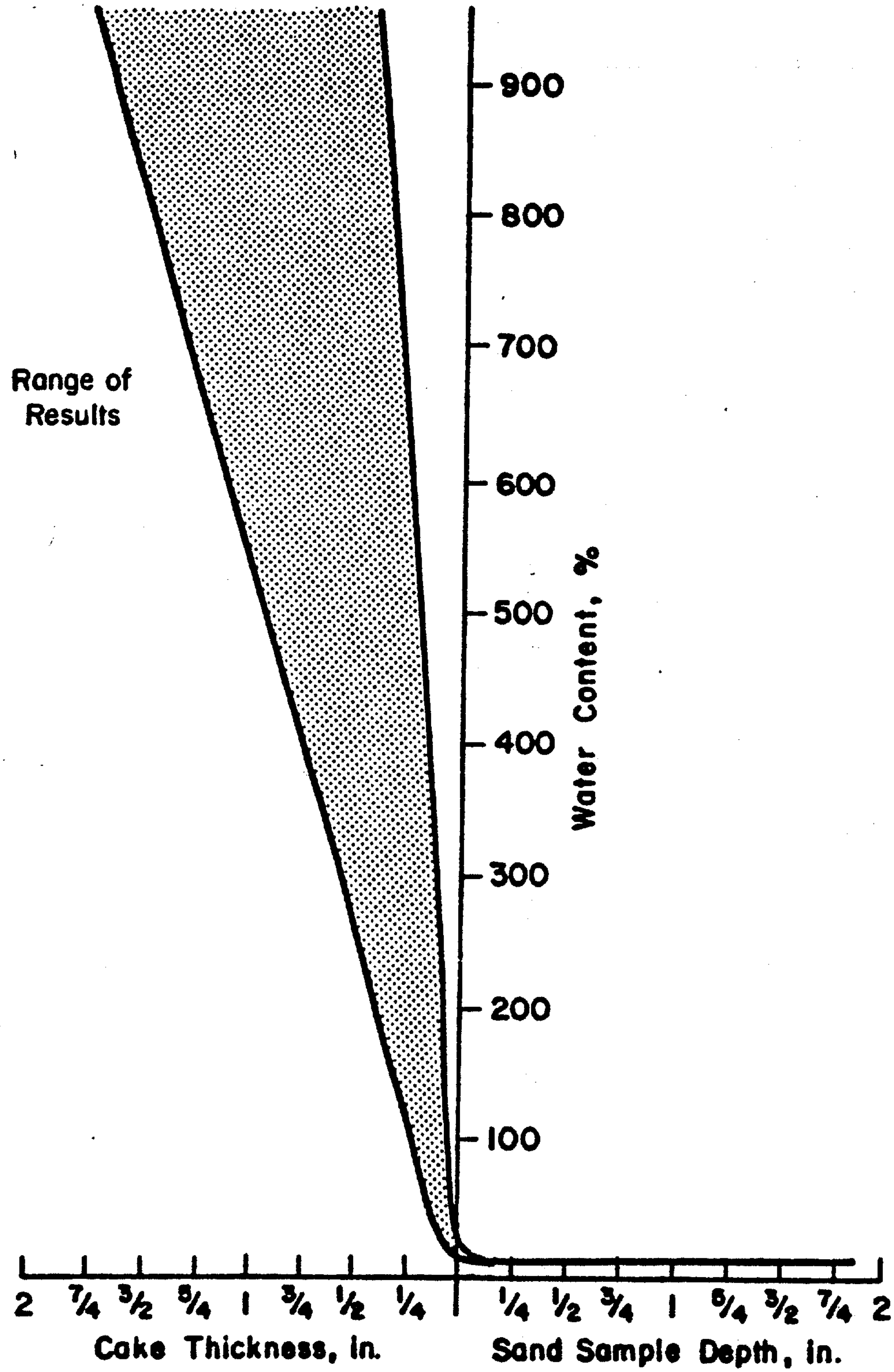
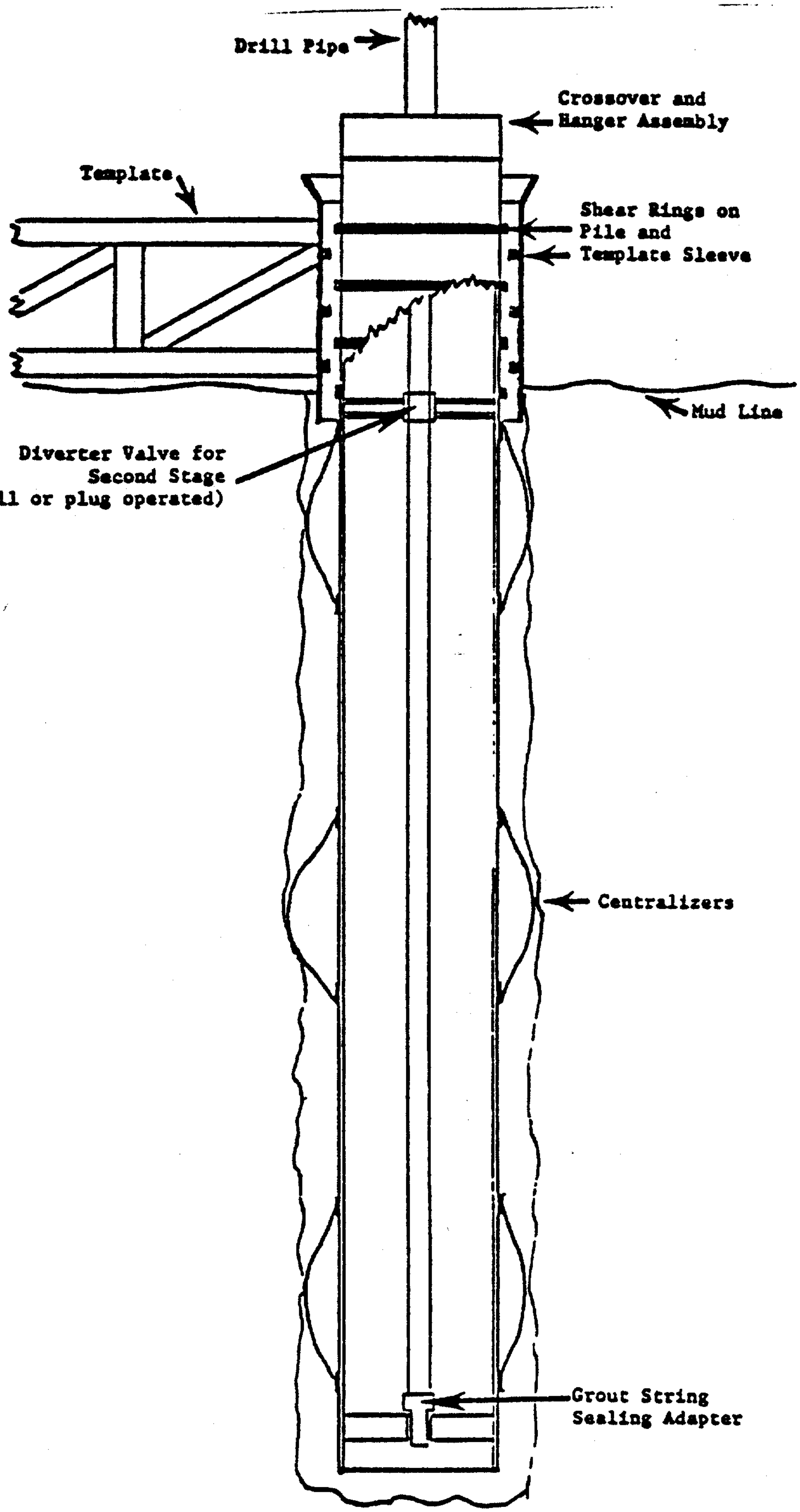
FIG. 3-4



EFFECT OF SEAFLOOR SCOUR ON DRILLING TEMPLATE STABILITY

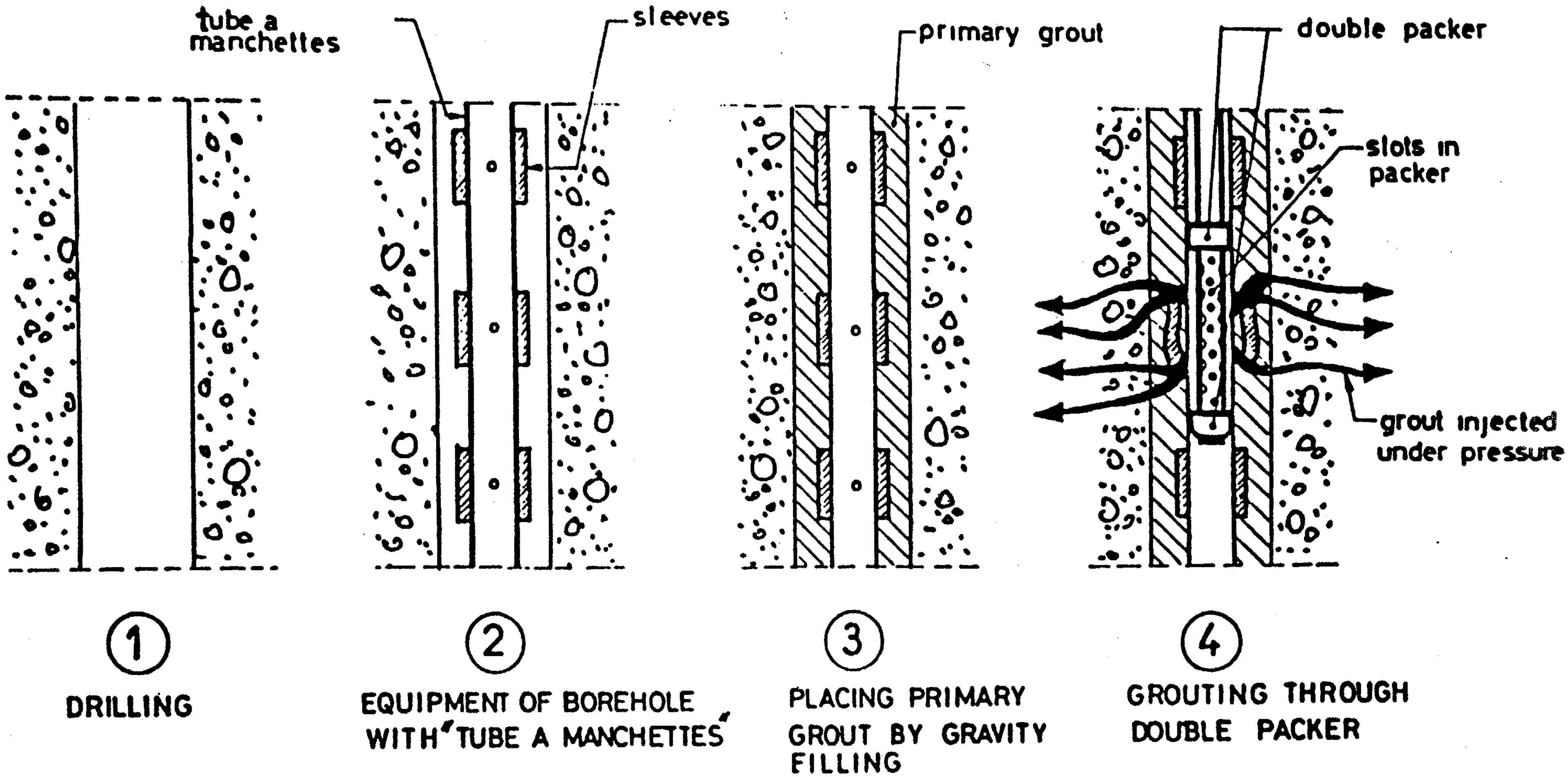


WATER CONTENT VS. FILTER CAKE THICKNESS
(After Tucker and Reese, 1984)



INNER STRING GROUTING FOR DRILLED HOLE PILES
(After Sutton et al., 1984)

JOB NO.



PRINCIPLES OF GROUTING WITH "TUBE A MANCHETTES"
(AFTER GOUVENOT, 1973)

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FIG. 4-2

REFERENCES

Aas, G., Lacasse, S, Lunne, T., and Hoeg, K. (1986), "Use of In-Situ Tests for Foundation Design on Clay", Proceedings, ASCE Conference on Use of In-Situ Tests in Geotechnical Engineering, GSP 6, Blacksburg, Virginia.

American Petroleum Institute (1986a), "API Recommendations for Planning, Designing and Constructing Fixed Offshore Platforms," API Report RP2A, 16th edition.

American Petroleum Institute (1987), "API Recommendations for Planning, Design and Constructing Fixed Offshore Platforms", 17th edition to be published.

American Railway Engineers Association (AREA) (1951), "Steel and Timber Pile Tests-West Atchafalaya Floodway - New Orleans, Texas & Mexico Railway, " Proceedings, 50th Annual Convention, AREA, Chicago, Ill., Vol. 52, pp. 149-202.

Angemeer, J., Carlson, E.D., Stroud, S., and Kurzeme, M. (1975), "Pile Load Tests in Calcareous Soils Conducted in 400 Feet of Water from A Semi-Submersible Exploration Rig," 7th Offshore Technology Conference, OTC 2311, Houston, Texas.

Arnold, P.G., Bea, R.G., Idriss, I.M., Reimer, R.B., Beebe, K.E., and Marshall, P.W. (1977), "A Study of Soil-Pipe Structure Systems in Severe Earthquakes", Proceedings, 9th Annual Offshore Technology Conference, OTC 2749, Houston, Texas, May.

Audibert, J.M.E., and Dover, A.R. (1982), "Discussion of Pile Load Tests: Cyclic Loads and Varying Load Rates," Proceedings, ASCE Journal of the Geotechnical Engineering Division, Vol. 108, No. GT3, March.

Baguelin, F., and Jezequel, J. (1972), "Etude experimentale du comportement de pieux sollicites" (Experimental Study of the Behavior of Piles Under Horizontal Loads), Bulletin, Liaison Laboratory, Ponts and Chaussess's.

Barden, L, and Mochkton, M.F. (1970), "Tests on Model Pile Groups in Soft and Stiff Clay," Geotechnique, Vol. 20, pp. 94-96, March.

Barthelemy, H. (1978a), "The MGBH-More Grout in the Borehole", unpublished patent, application by Solmarine, Nanterre, France.

Barthelemy, H. (1978b), "Bore Hole Pressure Control System", unpublished, patent application by Solmarine, Nanterre, France.

Bea, R.G., and Audibert, J.M.E. (1979), "Performance of Dynamically Loaded Pile Foundations", Proceedings, 2nd International Conference on Behavior of Offshore Structures (BOSS), Paper No. 68, London.

Bogard, D., and Matlock, H. (1979), "A Model Study of Axially Loaded Pile Segments, Including Pore Pressure Measurements," Report to the American Petroleum Institute, University of Texas, May.

Bogard, J.D., and Matlock, H. (1983), "Procedures for Analysis of Laterally-Loaded Pile Groups in Soft Clay," Proceedings, ASCE Specialty Conference on Geotechnical Engineering, Austin, Texas.

Briaud, J.L. and Felio, G. (1986), "Analysis of Existing Cyclic Vertical Load Tests for Piles in Clay," Proceedings, 18th Offshore Technology Conference, OTC 5224, Houston, Texas.

Briaud J.L., and Terry, T.A. (1986), "Rate Effect for Vertical and Horizontal Pile Response," Proceedings, 3rd International Conference on Numerical Methods in Offshore Piling, Nantes, France, May.

Broms, B. (1964a), "Lateral Resistance of Piles in Cohesionless Soils," Journal of the Soil Mechanics and Foundations Division, ASCE, pp. 123-156, May.

Broms, B. (1964b), "Lateral Resistance of Piles in Cohesive Soils," Journal of the Soil Mechanics and Foundations Division, ASCE, pp. 27-63, March.

Broms, B. (1965), "Design of Laterally Loaded Piles," Journal of the Soil Mechanics and Foundations Divisions, ASCE, pp. 79-99, May.

Brown, H.D., Grijalva, V.E., and Raymer, L.L. (1970), "New Developments in Sonic Wave Train Display and Analysis in Cased Holes", SPWLA 11th Annual Logging Symposium, May 3-6.

Burland, J.B. (1973), "Shaft Friction of Piles in Clay, A Simple Fundamental Approach," Ground Engineering, Volume 6, No. 3, pp. 35-40.

Bustamante, M., and Gouvenot, D. (1983), "Grouting: A Method Improving the Bearing Capacity of Deep Foundations", Int. Conf. on Soil Mechanics and Foundation Engineering, Helsinki.

Callis, C., Knox, C., Sutton, D., and Wiley, S. (1979), "An Assessment of Grouting Materials, Placement Methods and Monitoring Equipment for Offshore Structures", Proceedings, Offshore Technology Conference, OTC 3671, Houston, Texas, May.

Campbell, K.J. (1984), "Predicting Offshore Soil Conditions," Proceedings, 16th Offshore Technology Conference, OTC 4692, Houston, Texas.

Campbell, K.J., Hooper, J.R., and Prior, D.B. (1986), "Engineering Implications of Deepwater Geologic and Soil Conditions, Texas-Louisiana Slope," 18th Offshore Technology Conferenace, OTC 5105, Houston, Texas.

Casagrande, A., and Wilson, S.D. (1951), "Effect of Rate of Loading on the Strength of Clays and Shales at Constant Water Content," Geotechnique, Vol. 2, No. 3, pp. 251-263.

Chandler, R.J. (1968), "The Shaft Friction of Piles in Cohesive Soils in Terms of Effective Stress," Civil Engineering and Public Works Review, January.

Collipp, B.G., and Hall, M. (1984), "7,500 ft: A New Record Deepwater Drilling Program", Proceedings, Offshore Technology Conference, OTC 4746, Houston, Texas, May.

Cooke, C.E. Jr., Kluck, M.P., and Medrano, R. (1982), "Field Measurements of Annular Pressure and Temperature During Primary Cementing", Soc. of Petroleum Eng. of AIME, SPE 11206, 57th Ann. Fall Tech. Conf. and Exh., New Orleans, LA., Sept. 26-29.

Cox W.R., and Reese, L.C. (1978), "Pullout Tests of Grouted Piles in Stiff Clays", Journal of Petroleum Technology, March.

Coyle, H.M., and Reese, L.C. (1966), "Load Transfer for Axially Loaded Piles in Clay," Journal Soil Mechanics and Foundation Division, ASCE, Volume 92, No. SM2.

Coyle, H.M., and Sulaiman, I.H. (1967), "Skin Friction for Steel Piles in Sand," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 93, No. SM6, November, pp. 261-278.

Datta, M., Gulhati, S.K., and Rao, G.V. (1980), "An Appraisal of the Existing Practice of Determining the Axial Load Capacity of Deep Penetration Piles in Calcareous Sands," Proceedings, 12th Offshore Technology Conference, OTC 3867, Houston, Texas.

Davidson, M.T., and Prakash, S. (1963), "A Review of Soil-Pile Behavior," HRB Record No. 39.

Demars, K.R., and Chaney, R.C. (1981), Geotechnical Properties, Behavior, and Performance of Calcareous Soils," ASTM Symposium, STP 777, Ft. Lauderdale, Florida.

de Ruiter, J., and Beringer, F.L. (1979), "Pile Foundation for Large North Sea Structures", Journal, Marine Geotechnology, Volume 3, No. 3.

de Ruiter, J., and Richards, A.F. (1982), "Marine Geotechnical Investigations, A Mature Technology," Proceedings, ASCE Conference on Geotechnical Practice in Offshore Engineering, Austin, Texas.

Dougherty, J.J. Jr., and Hunt, H.W. (1982), "Driven Piling in Calcareous Soils," ASTM Symposium on Geotechnical Properties, Behavior and Performance of Calcareous Soils, STP 777, Ft. Lauderdale, Florida.

Doyle, E.H., and Pelletier, J.H. (1985), "Behavior of a Large Scale Pile in Silty Clay," Proceedings, XI Int. Conference on Soil Mechanics and Foundation Engineering, San Francisco.

Dutt, R.N., and Cheng, A.P. (1984), "Frictional Response of Piles in Calcareous Deposits," Proceedings, 16th Offshore Technology Conference, OTC 4838, Houston, Texas.

Dutt, R.N., Moore, J.E., Mudd, R.W., and Rees, T.E. (1985), "Behavior of Piles in Granular Carbonate Sediments from Offshore Philippines," Proceedings, 17th Offshore Technology Confernce, OTC 4849, Houston, Texas.

Duvivier, S., and Henstock, P.L. (1979), "Installation of the Piled Foundations and Production Modules on Occidental's Piper A Platform", Proceedings, The Institution of Civil Engineers, Part 1, Vol. 66, pp. 407-436, August.

Earth Technology Corporation, The (1983a), "State-of-the-Art Review in Foundation Design and Analysis for Tension Leg Platforms and Vertically Moored Platforms-Final Report", technical report prepared for Sandia National Laboratories, Albuquerque, NM, Report No. SAND 83-7120, May.

Edwards, M.R. (1978), "Platform Construction from an Adjacent Semi-submersible Vessel", Jour. of Petroleum Technology, pp. 1679-1687, December.

Ehlers, C.J., and Ulrich, E.J., Jr. (1977), "Design Criteria for Grouted Piles in Sand", Proceedings, 9th Offshore Technology Conference, OTC 2941, Houston, Texas.

Engeling, D.E., and Reese, L.C. (1974), "Behavior of Three Instrumented Drilled Shafts Under Short Term Axial Loading," Research Report 176-3, Center for Highway Research, The University of Texas at Austin, May.

Esrig, M.E., and Kirby, R.C. (1979), "Advances in General Effective Stress Methods for the Prediction of Axial Capacity for Driven Piles in Clay," Proceedings, 11th Offshore Technology Conference, Houston, Texas.

Farr, J.S., and Aurora, R.P. (1981), "Behavior of an Instrumented Pier in Gravelly Sand", Proceedings, ASCE Conference on Drilled Shafts and Caissons, pp. 53-65, St. Louis, Missouri.

Fleming, W.G.D., and Slivinski, Z. (1974), "Practical Considerations Affecting the Construction of Diaphragm Walls", Diaphragm Walls and Anchorages Conference, Institution of Civil Engineers, London, pp. 1-10.

Felio, G. (1985), "Influence of Cyclic Loading on Axially Loaded Piles in Clay," A Dissertation submitted to the Graduate College of Texas A&M University, May.

Flaate, K., and Selnes, P. (1978), "Side Friction of Piles in Clay," Norwegian Geotechnical Institute Publication No. 118.

Focht, J.A., and Koch, K.J. (1973), "Rational Analysis of the Lateral Performance of Offshore Pile Groups," Proceedings, 5th Offshore Technology Conference, OTC 1896, Houston, Texas.

Fragio, A.G., Santiago, J.L., and Sutton, V.J. (1985), "Load Tests on Grouted Piles in Rock," Proceedings, 18th Offshore Technology Conference, OTC 4851, Houston, Texas.

Froelich, B., Pittman, D., and Seeman, B. (1981), "Cement Evaluaton Tool - A New Approach to Cement Evaluation", Soc. of Petroleum Eng. of AIME, 56th Annual Fall Technical Conference and Exhibition, APE 10207, San Antonio, Texas, October 5-7.

Gates, W.E., Marshall, P.W., and Mahin, S.A. (1977), "Analytical Methods for Determining the Ultimate Earthquake Resistance of Fixed Offshore Structures", Proceedings, 9th Offshore Technology Conference, OTC 2954, Houston, Texas.

Gollwitzer, L.H., and Masson, J.P. (1982), "The Cement Bond Tool", SPWLA 23rd Annual Logging Symposium, July 6-9.

Gouvenot, D. (1973), "Essais en France et a L'etranger sur le Frottement Lateral en Fondations. Ameliorations par Injection", Travaux, 464, November.

Gouvenot, D., and Gabaix, J.C. (1975), "A New Foundation Technique Using Piles Sealed by Cement Grout Under High Pressure", Proceedings, and Seventh Annual Offshore Technology Conference, OTC 2310, May 5-8.

Gouvenot, D., and Bustamante, M. (1976), Essais de Pieux de Haute Capacite Scelles par Injection Sous Haute Pression", Proceedings, 6th European Conference Soil Mechanics, Vienne.

Grosch, J.J., and Reese, L.C. (1980), "Field Tests of Small-Scale Pile Segments in a Soft Clay Deposit Under Repeated Axial Loading," Proceedings, 12th Offshore Technology Conference, OTC 3869, Houston, Texas

Hagenaar, J., Chandrasekhar, C.S. and Waanders, A.J. (1986), "Lateral Loading Tests on Large Diameter Steel Pipe Piles Installed in Carbonate Rock and Soils," Proceedings, 19th Offshore Technology Conference, OTC 5146, Houston, Texas.

Halliburton Services (1984), "Model 73 Density Recorder System", Cementing Services technical data sheet No. C-1228.

Hanna, T.H. (1982), "Foundations in Tension-Ground Anchors", 1st edition, co-published by Trans-Tech Publ. and McGraw-Hill Book Comp.

Hart, H.J., White, G.J., and Van Fischer, E.L. (1985), "Hutton TLP Marine Operations", Proceedings, 17th Offshore Technology Conference, OTC 4910, Houston, Texas, May.

Hobbs, N.B. (1977), "Behavior and Design of Piles in Chalk - An Introduction to the Discussion of the Papers on Chalk," Institution of Civil Engineers, Thomas Telford, Ltd., London.

Hobbs, N.B., and Robins, P. (1977), "Compression and Tension Tests on Driven Piles in Chalk," Institution of Civil Engineers, Thomas Telford, Ltd., London.

Holloway, D.M., Clough, G.W., and Vesic, A.S. (1975), "The Mechanics of Pile-Soil Interaction in Cohesionless Soils," (FINAL) Contract Report S-75-5, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, December. (Also Duke Soil Mechanics Series No. 39).

Holmquist, D.V., and Matlock, H. (1976), "Resistance-Displacement Relationships for Axially-Loaded Piles in Soft Clay", Proceedings, 8th Offshore Technology Conference, OTC 2474, Houston, Texas.

Institution of Civil Engineers (1977), Piles in Weak Rock, Thomas Telford, Ltd., London.

Karlsrud, K. and Haugen, T. (1983), "Cyclic Loading of Piles and Pile Anchors: Field Model Tests," Final Report, Norwegian Geotechnical Institute, Report No. 40010-28, Oslo.

Kezdi, A. (1957), "The Bearing Capacity of Piles and Pile Groups," Proceedings, 4th International Conference of Soil Mechanics and Foundations Engineering, Vol. II, pp.46-51, London.

Kim, J.B., and Brungraber, R.J. (1976), "Full-Scale Lateral Load Tests of Pile Groups," Journal of the Geotechnical Engineering Division, ASCE, pp. 87-105, January.

King, R.W., Van Hooydonk, W.R., Kolk, H.F., and Windle, D. (1980), "Geotechnical Investigation of Calcareous Soils on the Northwest Shelf, Australia", Proceedings, 12th Offshore Technology Conference, OTC 3772, Houston, Texas.

Kraft, L.M., and Lyons, C.G. (1974), "State-of-the-Art: Ultimate Axial Capacity of Grouted Piles," Proceedings, 6th Offshore International Conference, OTC 2081, Houston, Texas.

Kraft, L.M. Jr., Cox, W.R., and Verner, E.A. (1981a), "Pile Load Tests: Cyclic Loads and Varying Load Rates", Jour. Geotechical Engineering Div., Vol. 107, No. GT1, pp. 1-19, January.

Kraft, L.M., Ray, R.P., and Kagawa, T. (1981b), "Theoretical T-Z Curves," Journal of Geotechnical Engineering Division, ASCE, Volume 107, GT11, pp. 1543-1561.

Kubo, K. (1968), "Experimental Study of the Behavior of Laterally Loaded Piles, Sols-Soils, Vol. 21, June.

Lam, I. and Martin, G. (1986), "Seismic Design of Highway Bridge Foundations," FHWA/RD-86/101 to 86/103.

Lee, H.J. (1984), "State of the Art: Laboratory Determination of the Strength of Marine Soils," ASTM Symposium on Strength Testing of Marine Sediments: Laboratory and In-Situ Measurements, STP 883, San Diego, California.

Lord, J.A. (1977), "A Comparison of Three Types of Driven Cast-In-Situ Piles in Chalk," Institution of Civil Engineers, Thomas Telford, Ltd., London.

Lu, B.T.D. (1986), "Axial Behavior and Capacity of Driven Piles in Calcareous Sands," Proceedings, 18th Offshore Technology Conference, OTC 5148, Houston, Texas.

Mallard, D.J., and Ballantyne, J.L. (1977), "The Behavior of Piles in Upper Chalk at Littlebrook D Power Station," Institution of Civil Engineers, Thomas Telford, Ltd., London.

Mansur, C.I., and Hunter, A.H. (1970), "Pile Tests-Arkansas River Project," Journal of the Soil Mechanics Foundation, ASCE Vol. 96, SM5, pp. 1545-1582, May.

Matlock, H. (1970), "Correlations for Design of Laterally Loaded Piles in Soft Clay", Proceedings, 2nd Offshore Technology Conference, OTC 1204, Houston, Texas.

Matlock, H., Meyer, P.L., and Holmquist, D.V. (1976), "AXCOL 3: A Program for Discrete-Element Solution of Axially Loaded Members with Linear or Nonlinear Supports", report to American Petroleum Institute, March.

Matlock, H., S.H.C. Foo, Tasi, C.F., and Lam P. (1978), "SPASM: A Dynamic Beam-Column Program for Seismic Pile Analysis with Support Motion", a Documentation Report to Chevron Oil Field Research Company from Fugro, Inc., Long Beach, California.

Matlock, H., and S.H.C. Foo (1979), "Axial Analysis of Piles Using a Hysteretic and Degrading Soil Model," Proceedings, Int. Conf. in Numerical Methods in Offshore Pilings, London, May.

Matlock, H., and Lam, P. (1980), "Design of Pile Foundations," Proceedings, International Symposium of Marine Soil Mechanics, Mexico, February.

Matlock, H., Bogard, D., and Lam, I.P. (1981), "BMCOL 76: A Computer Program for the Analysis of Beam-Columns Under Static Axial and Lateral Loading", Documentation Report to Ertec, Inc., June.

Matlock, H.M., Bogard, D., and Cheang, L. (1982a), "A Laboratory Study of Axially Loaded Piles and Pile Groups including Pore Pressure Measurements," Proceedings, 3rd International Conference on Behavior of Offshore Structures (BOSS), Cambridge, Mass.

Matlock, H., and Cheang, L. (1986), "Evaluation of Short, Large-Diameter Piles for Arctic Applications", to be published in Technology Assessment and Research Program for Offshore Minerals Operations, Department of the Interior, Minerals Management Service, Washington, D.C.

Matthews, W.R., and Kelly, J. (1967), "How to Predict Formation Pressure and Fracture Gradient, "Oil and Gas Journal, pp. 92, February.

McLamore, R.T., Ingram, W.B., and Porter, B.W. (1982), "Installing Anchor Piles for Offshore Rigs and Vessels", World Oil, February 1.

Menard, L. (1962), "Comportement d'une fondation profonde soumise a des efforts de renversement," Sol-Soils, No. 3, pp. 9-23.

Meyerhof, G.G. (1959), "Compaction of Sands and Bearing Capacity of Piles," Journal of Soil Mechanics Foundation Division, ASCE, Vol. 85, SM6, pp 1-29.

Meyerhof, G.G. (1976), "Bearing Capacity and Settlement of Pile Foundations," Journal of the Geotechnical Engineering Division, ASCE, Volume 102, No. GT3, pp. 197-228.

Mindlin, R.D. (1936), "Force at a Point in the Interior of a Semi-Infinite Solid," Physics, Volume 7.

Mueller, J.L., Audibert, J.M.E., and Bamford, S.R. (1986), "Grouted Instrumented Pile Study (GRIPS)," to be presented in the Mineral Management Service Technical Assessment and Research Program for Offshore Mineral Operations, Dept, of the Interior.

Murff, J.D. (1980), "Pile Capacity in a Softening Soil," Numerical and Analytical Methods in Geomechanics, Vol. 4, No. 2, 1980, pp. 185-189.

Nauroy, J.F., and Le Tirant, P. (1985), "Driven Piles and Drilled and Grouted Piles in Calcareous Sands," Proceedings, 17th Offshore Technology Conference, OTC 4850, Houston, Texas.

Ocean Industry (1986), "1986 Directory of Marine Drilling Rigs," September.

O'Neill, M.W., Ghazzaly, D.I., and Ha, H.B. (1977), "Analysis of Three Dimensional Pile Groups with Nonlinear Soil Response and Pile-Soil Pile Interaction," Proceedings, 9th Offshore Technology Conference, OTC 2838, Houston, Texas.

O'Neill, M.W., Hawkins, R.A. and Mahar, L.J. (1982), "Load Transfer Mechanisms in Piles and Pile Groups," Journal of Geotechnical Engineering Division, ASCE, Vol. 102, No. GT 12, December.

O'Neill, M.W. (1983), Group Action in Offshore Piles," Proceedings, Conference on Geotechnical Practice in Offshore Engineering, Austin, Texas, April.

Parry, R.H.G., and Swain, C.W. (1977), "Effective Stress Methods for Calculating Skin Friction on Driven Piles in Soft Clay," Ground Engineering, Volume 10, No. 3.

Peterson, L.M., Johnson, G.W., and Babb, L.V. (1986), "High Quality Sampling and In-Situ Testing for Deep Water Geotechnical Site Investigations", Proceedings, ASCE Conference on Use of In-Situ Tests in Geotechnical Engineering, GSP 6, Blacksburg, Virginia.

Petroleum Engineer International (1986), "ROVs Dive to Greater Depths of Technology", by Derrick Booth, December.

Poulos, H.G. (1971), "Behavior of Laterally Loaded Piles: II-Pile Groups," Journal of the Soil Mechanics and Foundation Division, ASCe, Volume 97, SM5.

Poulos, H.G. (1980), "Some Aspects of Skin Friction of Piles in Clay Under Cyclic Loading", National Technical Information Service, Publication No. 81-22673.

Poulos, H.G., and Davis, E.H. (1980), "Pile Foundation Analysis and Design," J. Wiley and Sons, New York.

Quiros, G.W., Young, A.G., Pelletier, J.H., and Chan, J.H-C. (1982), "Shear Strength Interpretation for Gulf of Mexico Clays," Proceedings, ASCE Conference on Geotechnical Practice in Offshore Engineering, Austin, Texas.

Rabinowitz, P.D., Garrison, L.F., Harding, B.F., Herrig, S.D., Kidd, R.B., Merrill, R., Meyer, A.W. and Olivas, R.E. (1986), "The Ocean Drilling Program After 1 Year of Drilling Operations", Proceedings, 18th Offshore Technology Conference, OTC 5184, Houston, Texas.

Randolph, M.F., and Murphy, B.S. (1985), "Shaft Capacity of Driven Piles in Clay," Proceedings, 17th Offshore International Conference, OTC 4883, Houston, Texas.

Reese, L.C. (1964), "Load vs. Settlement for an Axially Loaded Pile," Proceedings, Symposium on Bearing Capacity of Piles, Central Building Research Institute, Roorkee, India.

Reese, L.C., Hudson, W.R. and Vijayvergiya, V.N. (1969), "An Investigation of the Interaction Between Bored Piles and Soil," Proceedings, Seventh International Conference on Soil Mechanics and Foundations Engineering, Mexico City.

Reese, L.C., Cox, W.R., and Koop, F.D. (1974), "Analysis of Laterally Loaded Piles in Sand," Proceedings, 6th Offshore Technology Conference, OTC 2080, Houston, Texas.

Reese, L.C., Cox, W.R., and Koop, F.D. (1975), "Field Testing and Analysis of Laterally Loaded Piles in Stiff Clay", Proceedings, 7th Offshore Technology Conference, OTC 2312, Houston, Texas.

Reese, L.C., and Welch, C. (1975), "Lateral Loading of Deep Foundation in Stiff Clay", Journal, Geotechnical Engineering Division, ASCE Vol. 101, No. GT7, July.

Reese, L.C., Touma, F.T. and O'Neill, M.W. (1976), "Behavior of Drilled Piers Under Axial Loading," Journal ASCT, Geotechnical Engrg. Division, Vol. 102, No. G75, May.

Reese, L.C., and Wright, S.G. (1977), Drilled Shaft Manual, Volume I, Construction Procedures and Design for Axial Loading, Report to U. S. Dept. of Transportation, July.

Reese, L.C., and Allen, J.D. (1977), Drilled Shaft Manual, Volume II, Structural Analysis and Design for Lateral Loading, Report to U. S. Dept. of Transportation, July.

Reese, L.C., (1977), "Laterally Loaded Piles: Program Documentation", Proceedings, American Society of Civil Engineers, Volume 13, No. Gt4, April.

Richards, A.F., and Zuidberg, H.M. (1984), "In-Situ Determination of the Strength of Marine Soils," ASTM Symposium on Strength Testing of Marine Sediments: Laboratory and In-Situ Measurements, STP 883, San Diego, California.

Richardson, P. (1977), "Installing Pile Foundations", Petroleum Engineer International, February.

Richardson, P., and Boelsemo, C.R. (1978), "Self-Contained Pile Installation of an Offshore Platform", Proc. Oceanology International 78, London.

St. John, H.D. (1980), "A Review of Current Practice in the Design and Installation of Piles for Offshore Structures", Building Research Station, Dept. of Energy, Offshore Technology Paper No. 5, London.

Saffery, M.R., and Tate, A.P.K. (1961), "Model Tests on Pile Groups in a Clay Soil with Particular Reference to Behavior of the Group When it is Loaded Eccentrically," Proceedings, 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, pp. 129-134.

Sangrey, D.A. (1977), "Response of Offshore Piles to Cyclic Loading," Proceedings, Offshore Technology Conference, OTC Paper 2944.

Schlitt, H.G. (1951), "Group Pile Loads in Plastic Soils," Proceedings, Highway Research Board, Washington, D.C., Vol. 31, pp. 62-81.

Scripps Institute of Oceanography (1984), "Initial Core Description - Deep Sea Drilling Project, Leg 94, North Atlantic Ocean, June 17 - August 17, 1983", a report to National Science Foundation.

Searle, I.W., and Bartholomew, R.F. (1977), "The Behavior of Driven Cast-In-Situ and Bored Piles in Weathered Chalk," Institution of Civil Engineers, Piles in Weak Rock, Thomas Telford, Ltd., London, England.

Seed, H.B., and Reese, L.C. (1955), "Action of Soft Clay Along Friction Piles," Proceedings, ASCE, Vol. 81, Paper 842.

Semple, R.M., and Rigden, W.J. (1984), "Shaft Capacity of Driven Pipe Piles in Clay", Proceedings, Symposium on the Analysis and Design of Pile Foundations, San Francisco, California, October.

Settgast, R.H., and Sutcliffe, G. (1980), "Marine Pile Load Testing in Carbonate Rocks", 12th Offshore Technology Conference, OTC 3868, Houston, Texas.

Skempton, A.W., Yassin, A.A., and Gibson, R.E. (1953), "Theorie de la force portante des pieux," Annales de l'Institut Technique du Batiment et des Travaux Publics, Vol. 6, no. 63-64.

Sliwinski, Z.J. (1977), "The Effect of Bentonite Displacement Method of Construction on the Bearing Capacity of Piles and Diaphragm Wall Elements," London, October.

Soletanche Entreprise (1974), "Procedes Nouveaux de Fondations de Plateforms Offshore - Essais de Dunkerque, Rapport d'Injection", Paris, 19 June, ("New Processes for Offshore Platform Foundations - Dunkirk Tests, Injection Report", Paris, 19 June.

Sol Expert International (1973), "Etude des Fondations de Plateforme de Forage Offshore", Rapport de Synthese Annexe II, Liaison Pile/Insert Pile, Paris, 15 Mai ("Study of Foundations for Offshore Drilling Platforms", Summary Report, Annex II, Connection Pile/Insert Pile, Paris, 15 May.

Sowers, G.F., Martin, C.B, Wilson, L.L., and Fausold, M. (1961), "The Bearing Capacity of Friction Pile Groups in Homogeneous Clay from Model Studies," Proceedings, 5th International Conference on Soil Mechanics and Foundation Engineering, Vol. 2, pp. 155-159.

Stevens, J.B., and Audibert, J. (1979), "Re-examination of P-Y Curve Formulations for Clays," Proceedings, 11th Offshore Technology Conference, OTC 3402, Houston, Texas.

Sutton, D. and Gerke, R.R. (1984), Halliburton Services, technical communication.

Taylor, D.W., and Whitman, R.V. (1953), "The Behavior of Soils Under Dynamic Loads. 2. Interim Report on Wave Propagation and Strain-Rate Effect," MIT Dept. of Civil and Sanitary Engineering, Report AFSWP-117 to the Office of the Chief of Engineers, July.

Taylor, M.J. (1986), Brown & Root, technical communication.

Templeton, J.S., Murff, J.D., Klejbuk, L.W., and Goodwin, R.H. (1985), "Evaluating Soils and Hazards in the Mississippi Canyon," Proceedings, 17th Offshore Technology Conference, OTC 4964, Houston, Texas.

Tjelta, T.-I., Watt, P.H., and Senner, D.W.F. (1982), "Deep Water Soil Investigation: Planning to Results," Proceedings, ASCE Conference on Geotechnical Practice in Offshore Engineering, Austin, Texas.

Tomlinson, M.J. (1977), "Pile Design and Construction Practice", London, Viewpoint Publications.

Touma, F.T., and Reese, L.C. (1974), "Behavior of Bored Piles in Sand," Journal, Geotechnical Division, ASCE, Vol. 100, GT7, July.

Tucker, R.L., and Reese, L.C. (1984), "The Effects of Bentonite Slurry on Drilled Shafts," report to Texas State Dept. of Highways and Public Transporation by Center for Transportation Research, Univ. of Texas, July.

Vesic, A.S. (1961), "Bending of Beams Resting on Isotropic Elastic Solid, Journal of Engineering Mechanics Division, ASCE, Vol. EM 2-87, pp. 35-53.

Vesic, A., and Johnson, W. (1963), "Model Studies of Beams Resting on a Silt Subgrade," Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 89, No. SM1, pp. 1-31.

Vesic, A.S. (1967), "A Study of Bearing Capacity of Deep Foundations," Final Report, Proj. B-189, School of Civil Engineering, Georgia Inst. Tech., Atlanta, Georgia.

Vesic, A.S. (1974), Unpublished Lecture Notes for CE 235: "Foundation Engineering," Duke University, Durham, NC.

Vesic, A.S. (1975), "Principles of Pile Foundation Design," Duke University School of Engineering, Soil Mechanics, Series No. 38.

Vesic, A.S. (1977), "Design of Pile Foundation, National Cooperative Highway Research Program," Synthesis of Highway Practice 42, National Research Council, Washington, D.C.

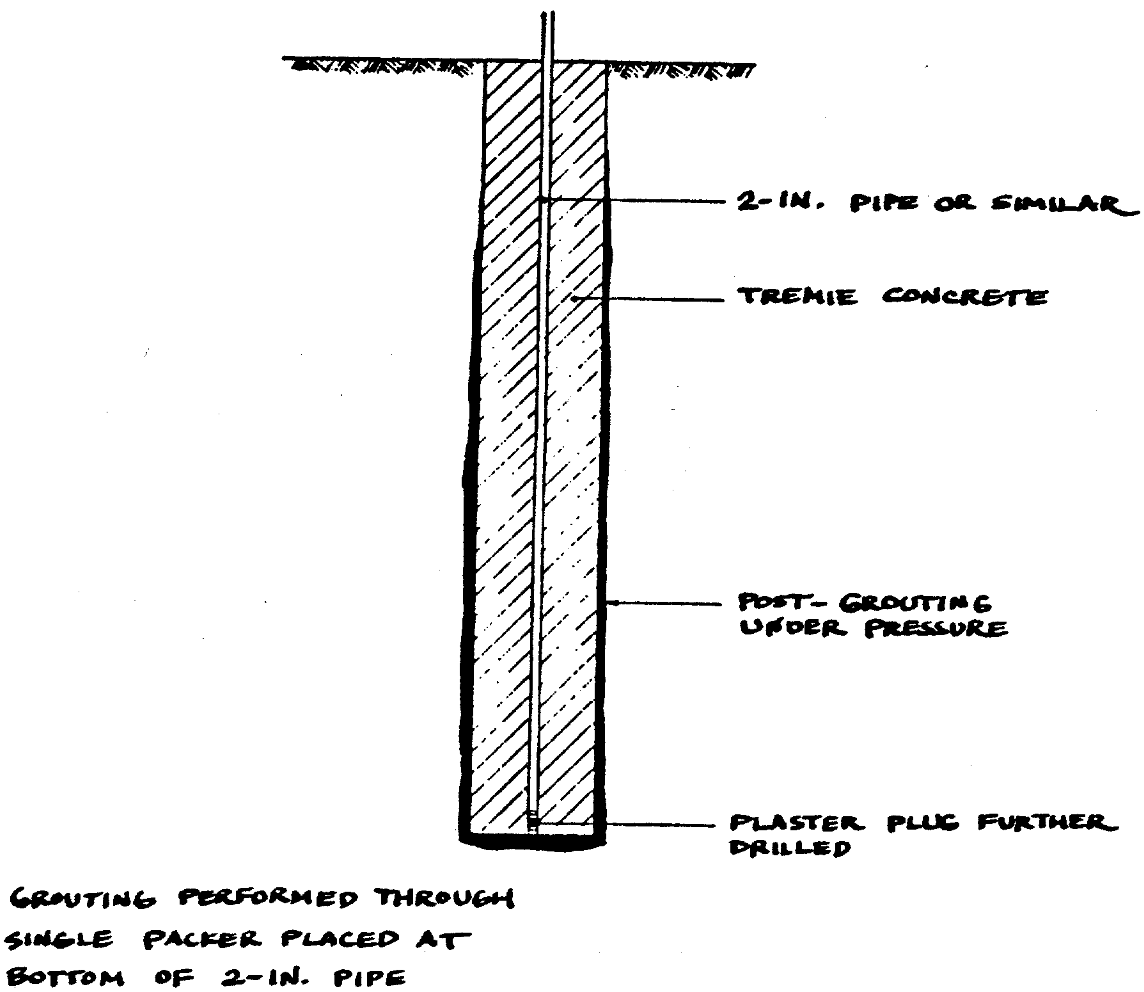
Vijayvergiya, V.N., and Focht, J.A. (1972), "A New Way to Predict Capacity of Piles in Clay," Proceedings, 4th Offshore Technology Conference, OTC 1718, Houston, Texas.

Vijayvergiya, V.N. (1977), "Load-Movement Characteristics of Piles," Proceedings, Port 1977 Conference, Long Beach, California.

Whitaker, T. (1957), "Experiments with Model Piles in Groups," Geotechnique, Vol. VII.

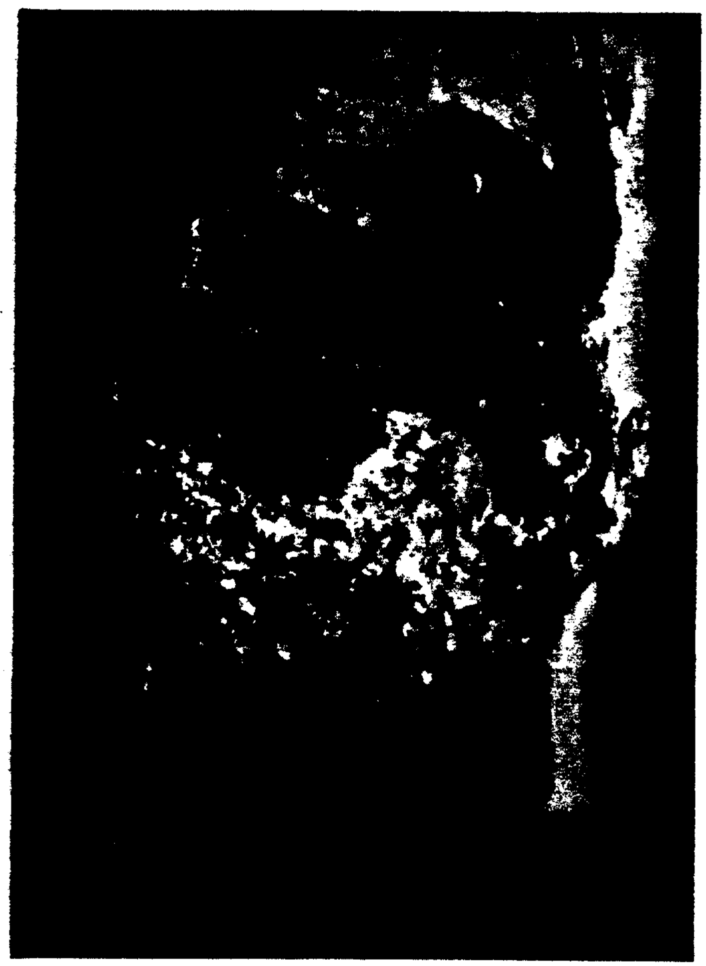
Young, A.G., Sullivan, R.A., and Rybicki, C.A. (1978), "Pile Design and Installation Features of the Thistle Platform," EUR 12, Proceedings, European Offshore Petroleum Conference and Exhibition, London, October.

CROSS-SECTION OF GROUT BULB FORMED BY HIGH-PRESSURE MULTIPLE INJECTION GROUTING
(After Gouvenot, 1978)



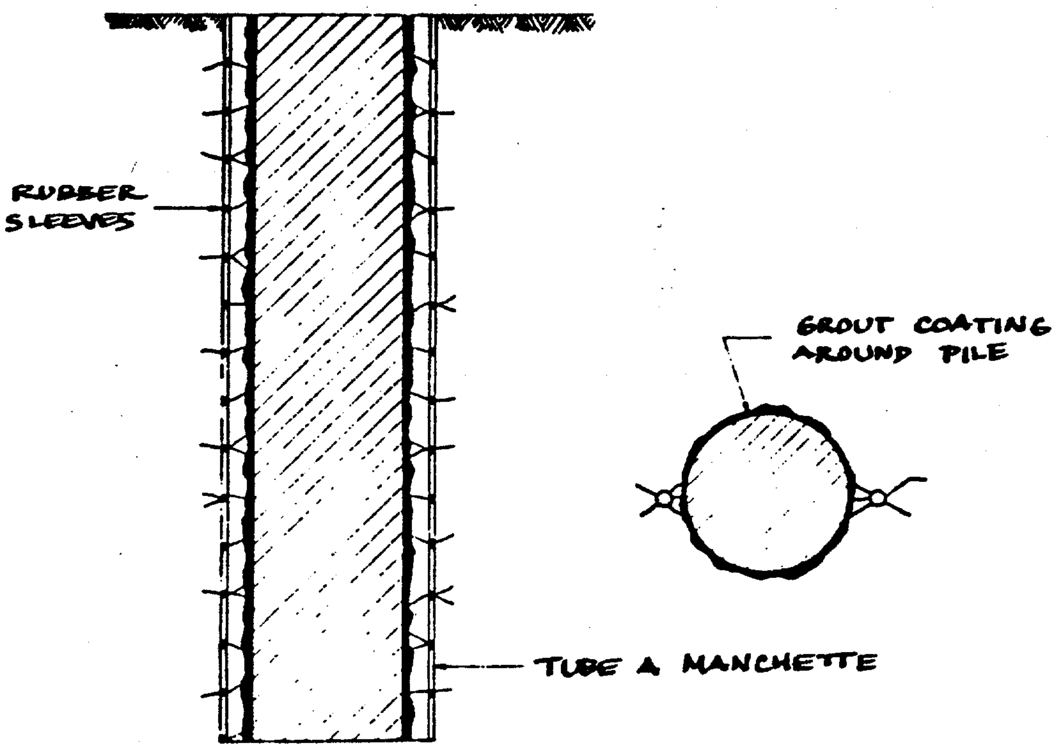
TOE GROUTING
(Modified after Bustamante and Gouvenot, 1983)

(a)

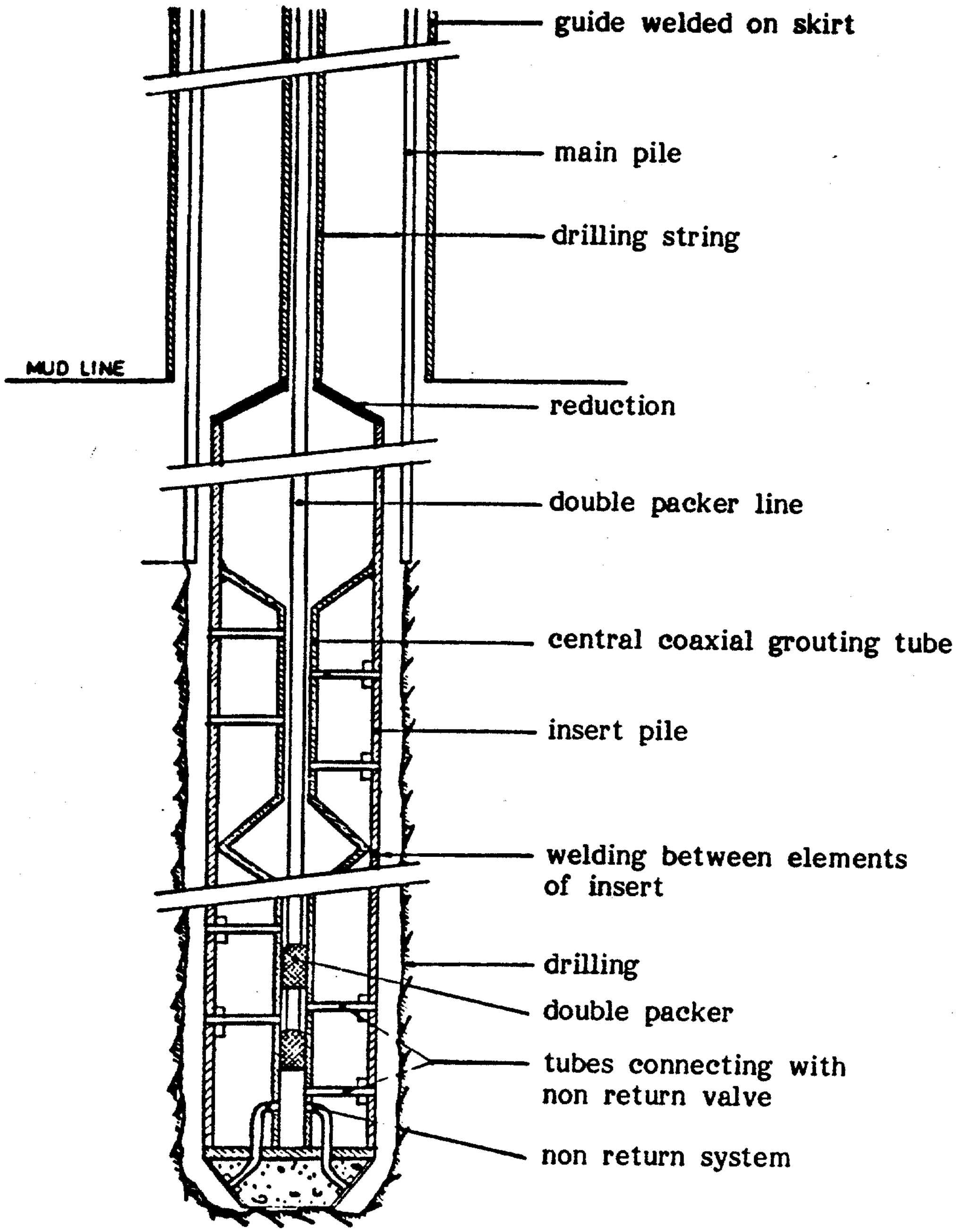


(b)

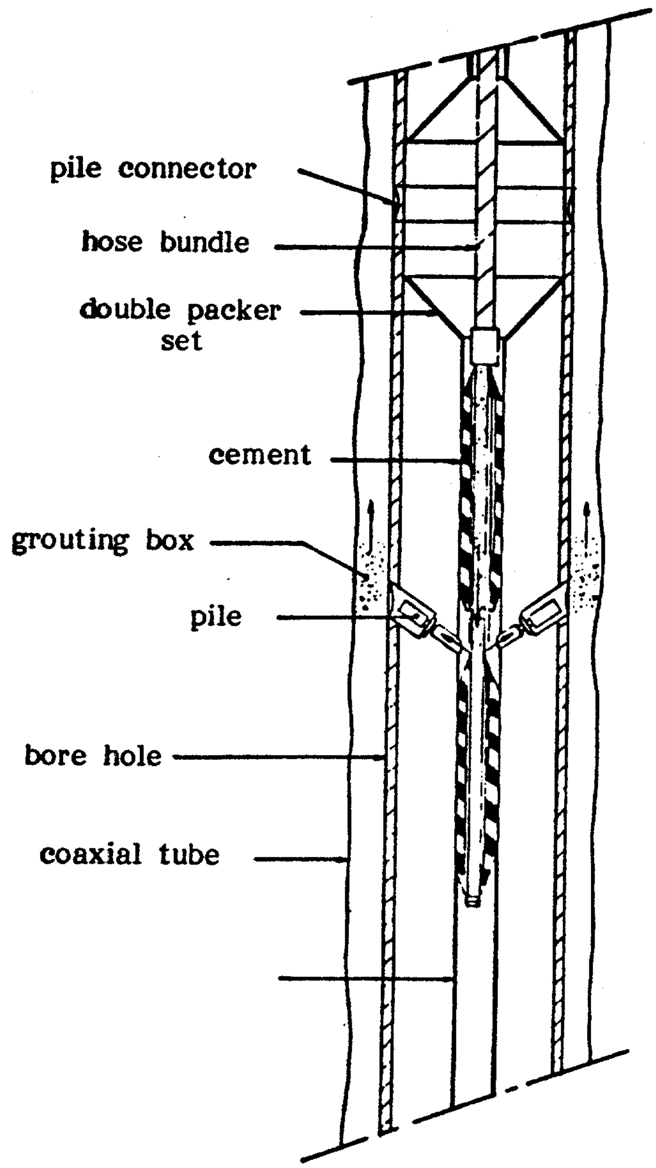
BORED PILES EXTRACTED FOLLOWING LOAD TESTING
(After Soletanche Enterprise, 1974)



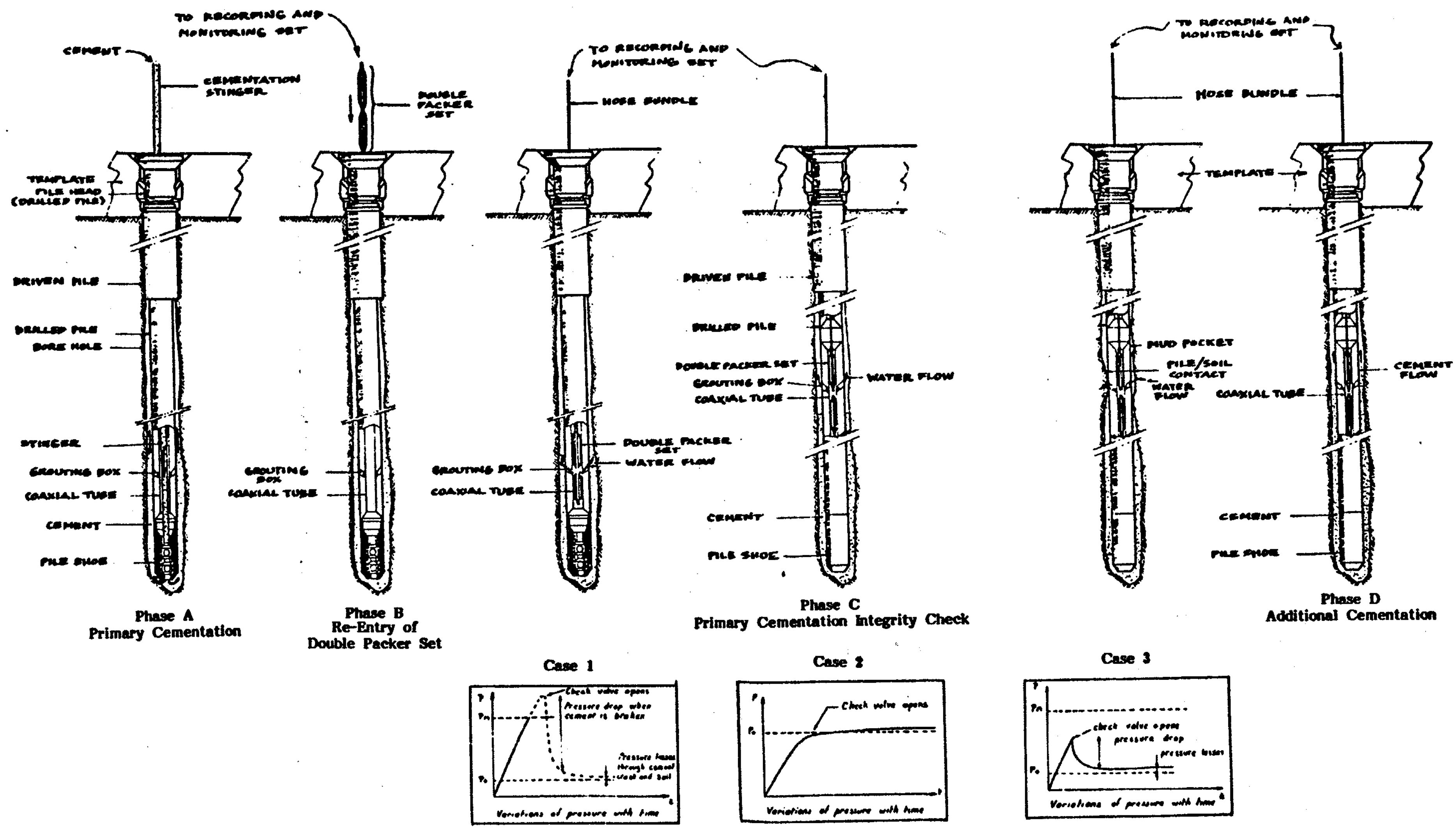
TREMIE CONCRETE PILES - GROUTING AROUND SHAFT
(Modified after Bustamante and Gouvenot, 1983)



CENTRAL GROUTING SYSTEM
(Modified after Soletanche Enterprise, 1974)



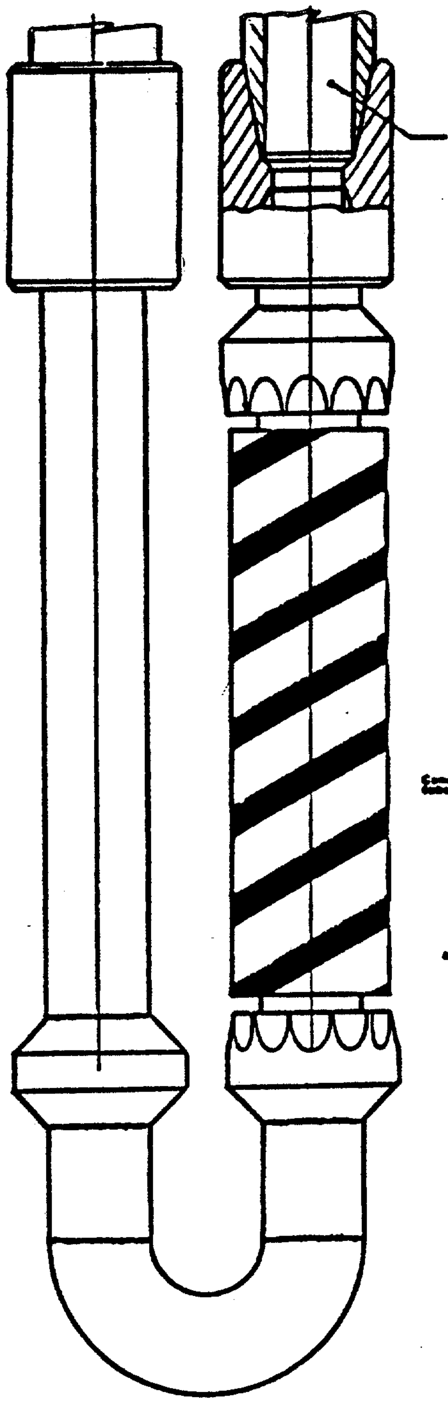
CEMENTATION THROUGH GROUTING BOXES
(Modified after Soletanche Enterprise, 1974)

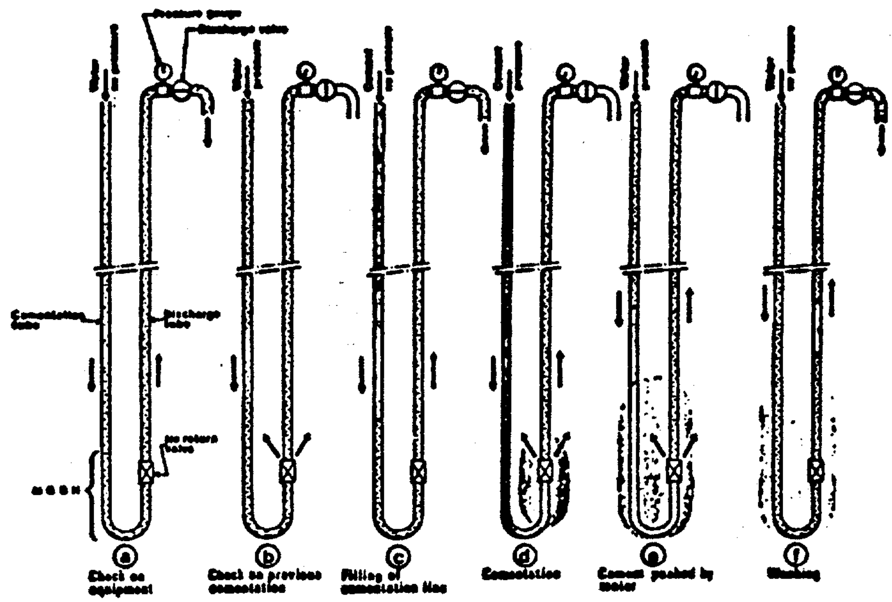


DRILLED AND GROUTED PILE CENTRAL GROUTING SYSTEM
(After Soletanche Enterprise, 1974 and Barthelemy, 1978a)

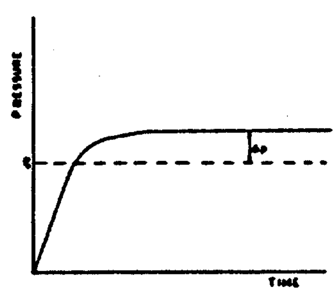
The Earth Technology Corporation

FIG. 4-9





CHECK ON PREVIOUS GROUTING

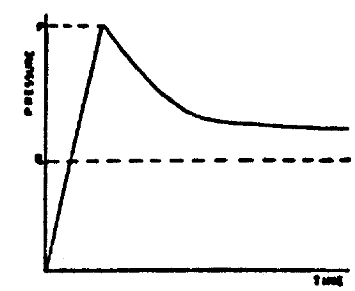


Typical reaction when the annulus is filled with mud or fluid grout

The pressure raises until the valve opens, then it remains constant.

This opening pressure is the sum of Po, only due to pressure losses through the valve, and Δp, function of the hydrostatic back pressure in the annulus.

The value of Δp shows wether the annulus is filled with mud (primary cementation failed) or with unset grout.

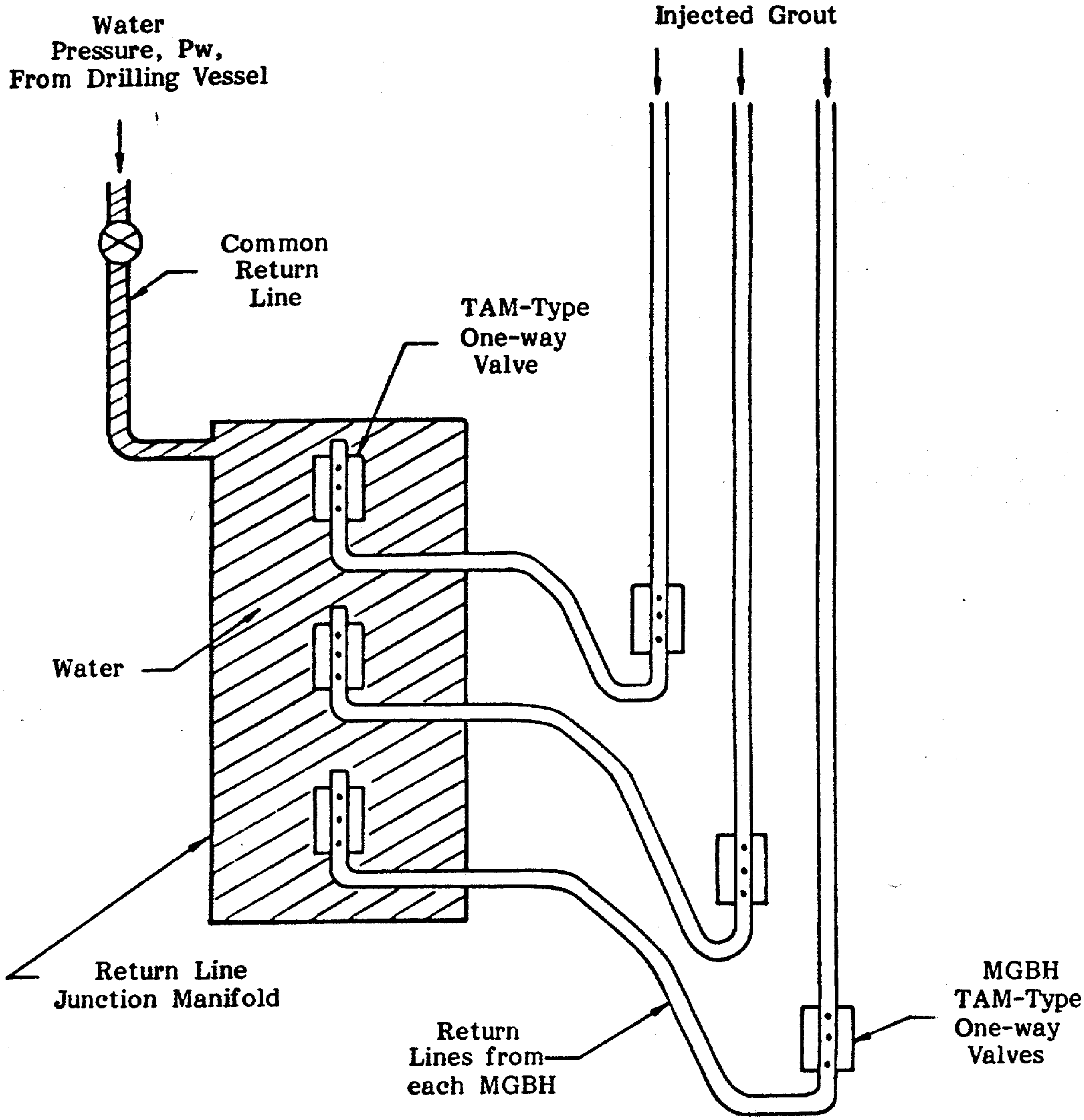


In the case shown above, the valve opened under a high pressure, followed by a pressure drop.

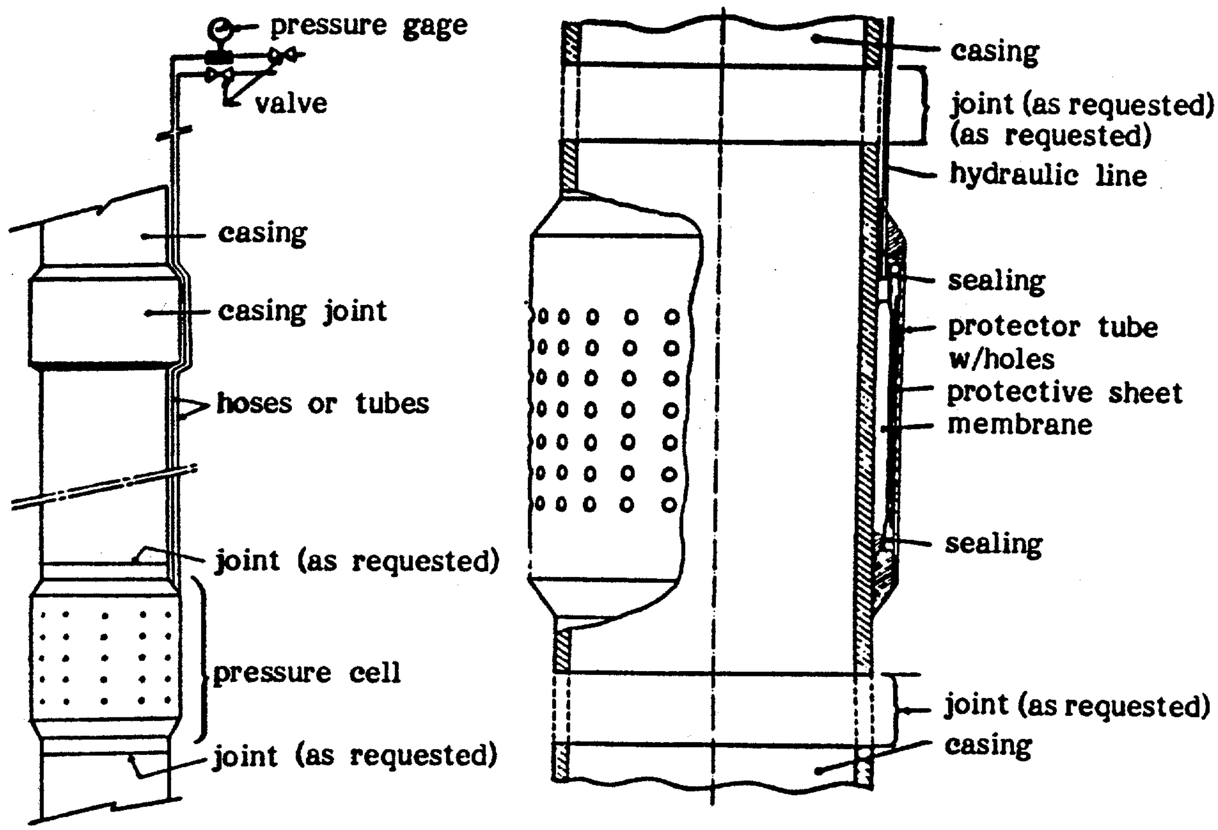
It shows that grout had set at that level, and had to be fractured.

The value of P is again an important parameter to analyse, as it is a direct function of the grout strength.

M.G.B.H. GROUTING SYSTEM
(Modified after Barthelemy, 1978a)

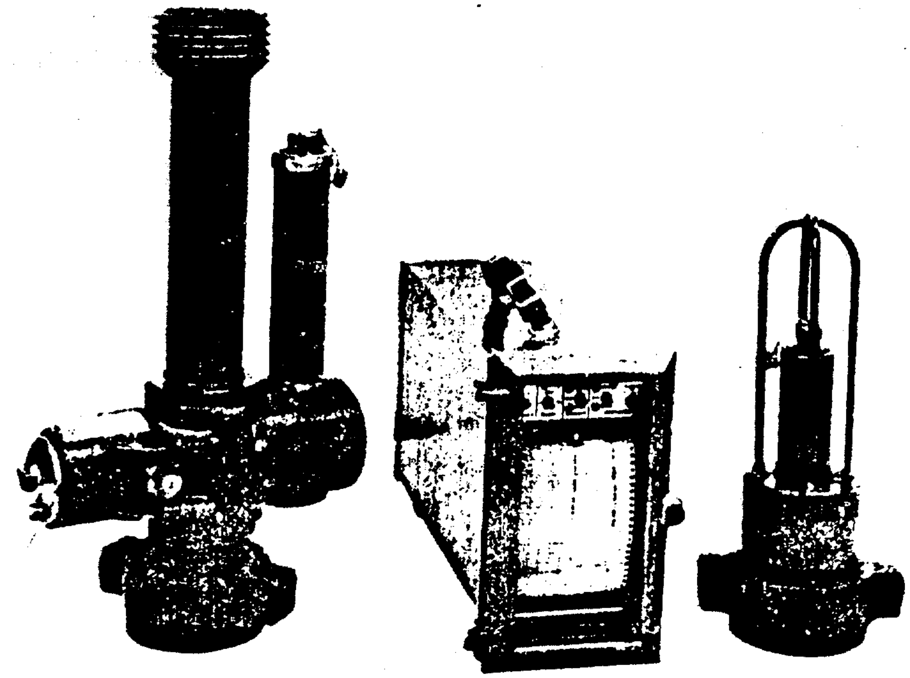


MGBH SYSTEM WITH COMMON RETURN LINE
(After Barthelemy, 1978a)

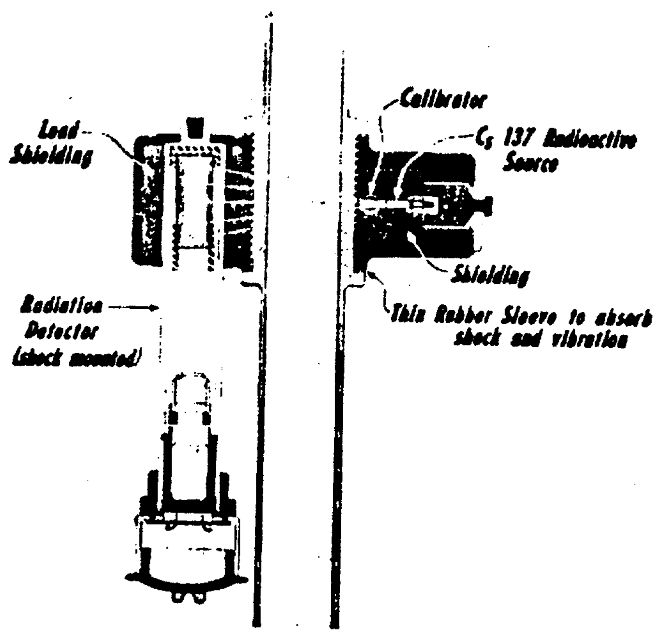


BOREHOLE PRESSURE CONTROL SYSTEM
(Modified after Barthelemy, 1978b)

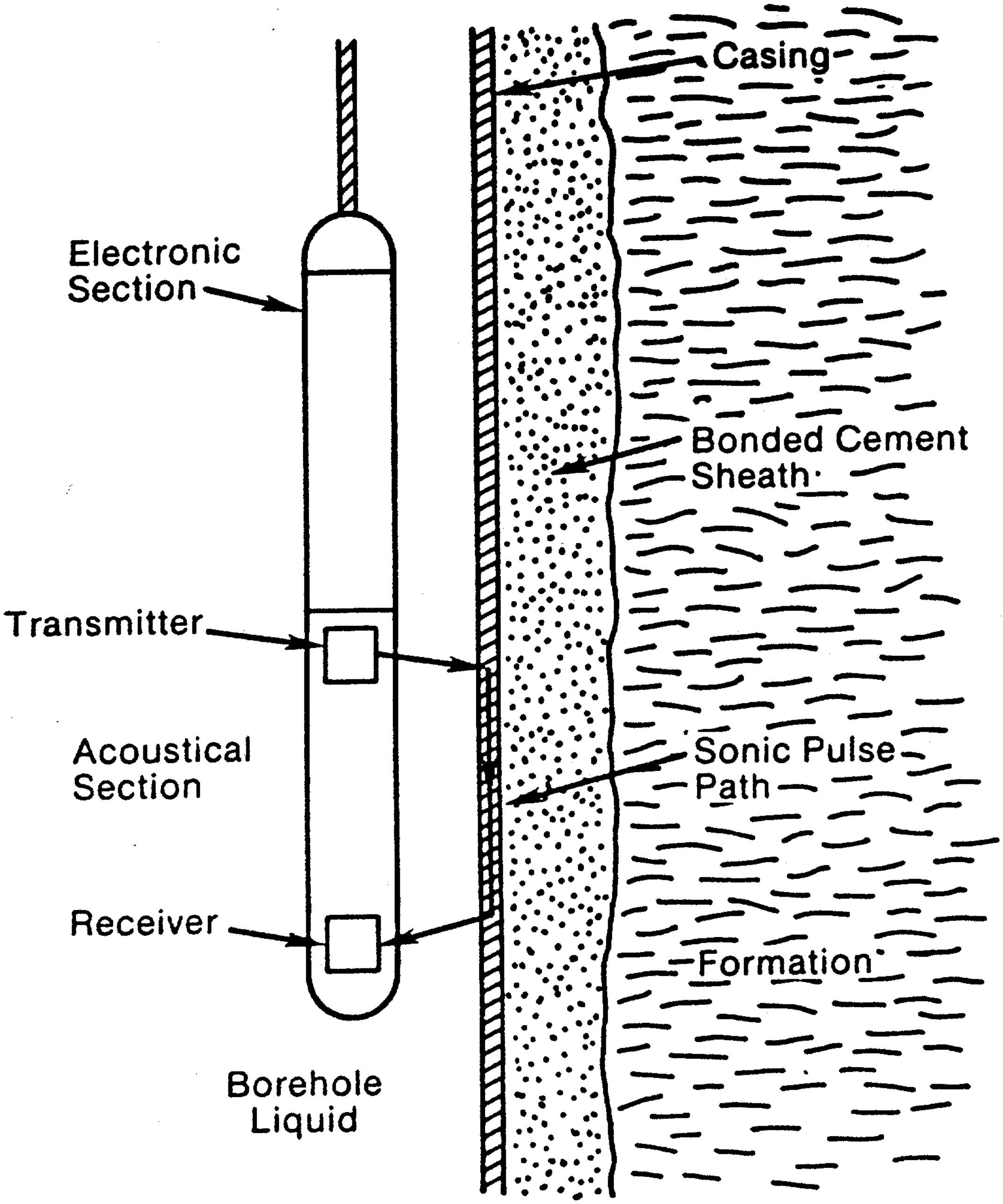
a) Density Recorder System



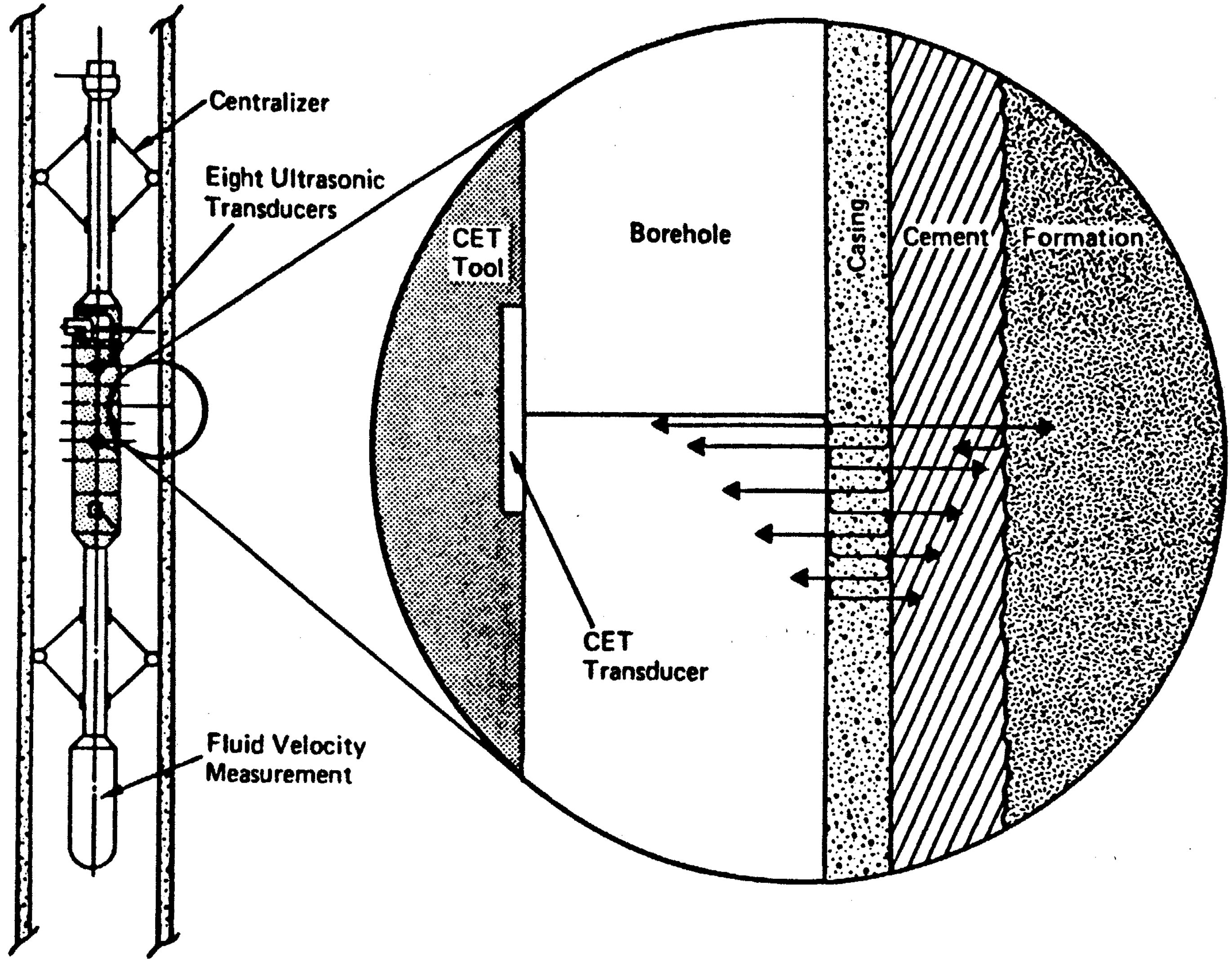
b) RADIOACTIVE DENSOMETER



DENSITY RECORDING SYSTEM AND RADIOACTIVE DENSOMETER
(After Halliburton Services, 1984)



SCHEMATIC OF CEMENT BOND LOGGING TOOL
(After Bruckdorfer et al., 1983)



SCHLUMBERGER CEMENT EVALUATION
TOOL OPERATION
(After Janele, 1984)

TABLE 1

BONDING PROPERTIES OF VARIOUS PIPE FINISHES
Cement — API Class A Cement
Water — 5.2 Gallons Per Sack
Curing Temperature — 80°F.
Curing Time — 1 Day
Casing Size — 2 Inch inside 4 Inch

TYPE OF FINISH	BOND STRENGTH		
Steel	Shear PSI	Hydraulic PSIG	Gas PSIG
New (Mill Varnish)	74	200-250	15
New (Varnish Chemically Removed)	104	300-400	70
New (Sandblasted)	123	500-700	150
Used (Rusty)	141	500-700	150
New (Sandblasted - Resin-Sand Coated)	2400	1100-1200	400+
Plastic			
Filament Wound (Smooth)	79	210	—
(Rough)	99	270	—
Centrifugally Cast (Smooth)	81	220	—
(Rough)	101	310	—

TABLE 2

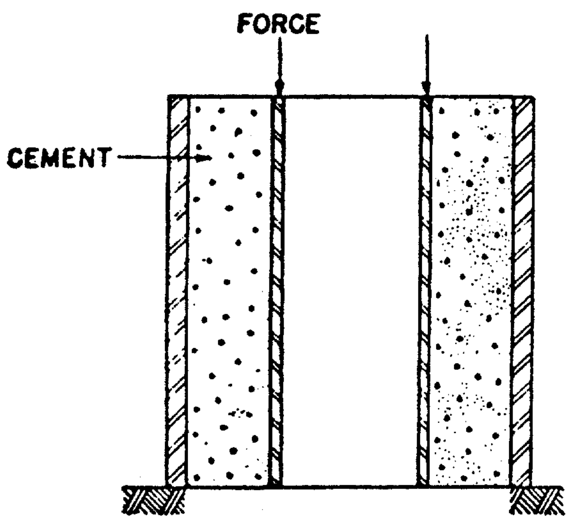
BOND STRENGTH
Casing Condition — New Mill Varnish
Cement — API Class A Cement
Water — 5.2 Gallons Per Sack
Curing Temperature — 100°F.

Time Days	Hydraulic (Water-PSIG)	Gas (Nitrogen-PSIG)	Shear PSI
1	200	15	74
2	170	10	63
3	210	30	72
4	250	40	74
5	270	55	76

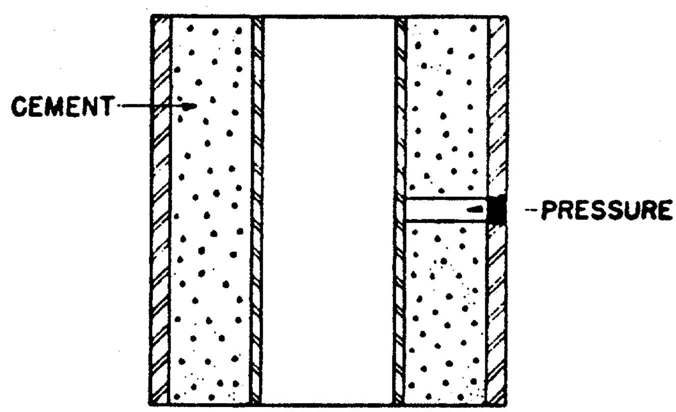
TABLE 3

BONDING PROPERTIES ON SURFACE WETTING OF PIPE
Cement — API Class A Cement
Water — 5.2 Gallons Per Sack
Curing Temperature —80°F.
Curing Time — 1 Day
Casing Size — 2 Inch inside 4 Inch
Type Casing — Used

Type of Mud	Shear Bond PSI
None	141
Water Base	97
Inverted Oil Emulsion	66
Oil-Base	63

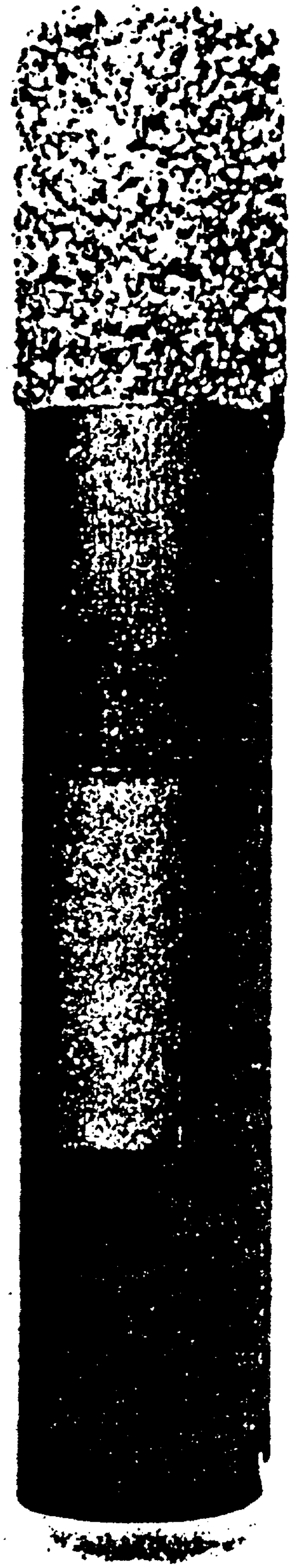


SHEAR BOND TEST



HYDRAULIC BOND TEST TO PIPE

PILE-GROUT BOND STRENGTH TEST RESULTS (AFTER EVANS AND CARTER, 1964)



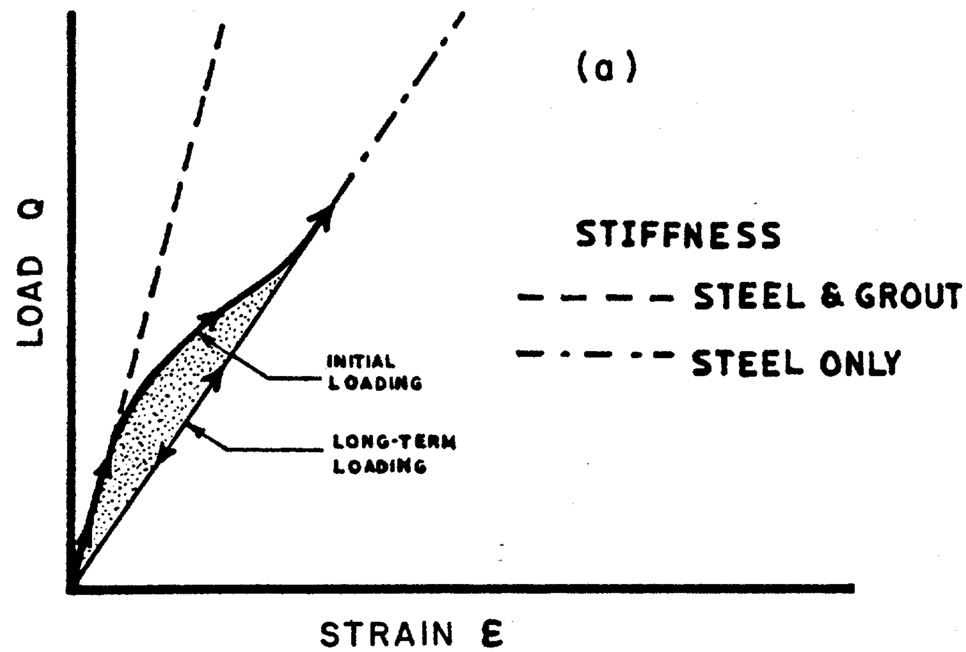
	Shear Bond PSI	Hydraulic Bond PSIG	Gas Bond PSIG
RESIN-SAND COATED	2400	1100-1200	400+
RUSTY	141	500-700	150-250
SAND BLASTED	123	500-700	150-250
MILL VARNISH	79	200	10-20

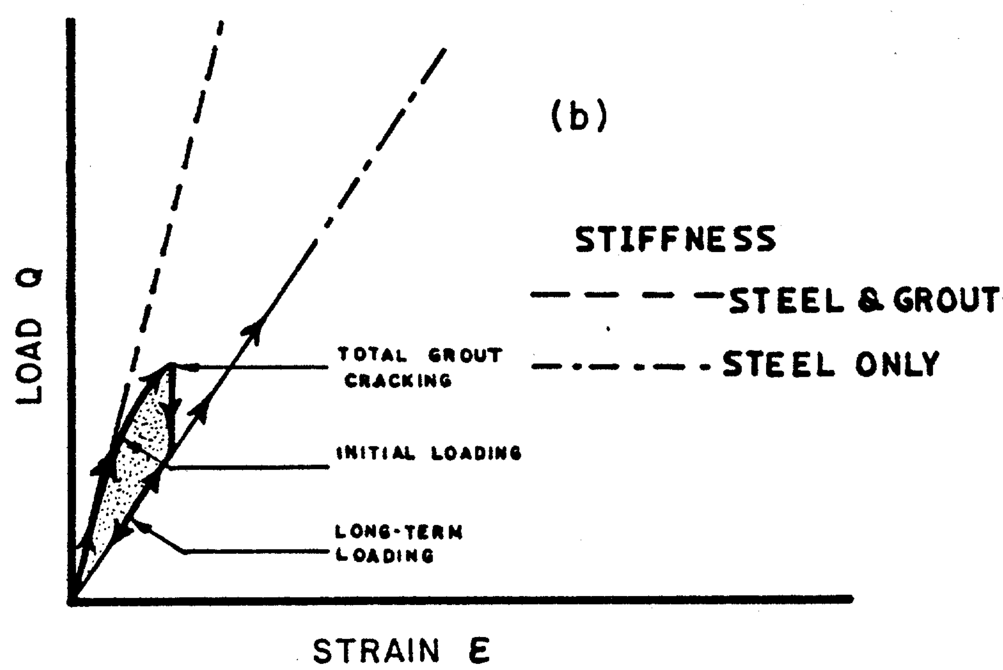
PILE-GROUT BOND STRENGTH AS A FUNCTION OF PILE SURFACE FINISH (AFTER EVANS AND CARTER, 1964)

Grout Type	Bond Strength, (psi)
Cement (w/c = 0.4)	430 to 580
Clinker cement	1,300
Clinker cement + acrylic	1,100
Polyester resin	430 to 1,750
Epoxy resin	2,200 to 2,750

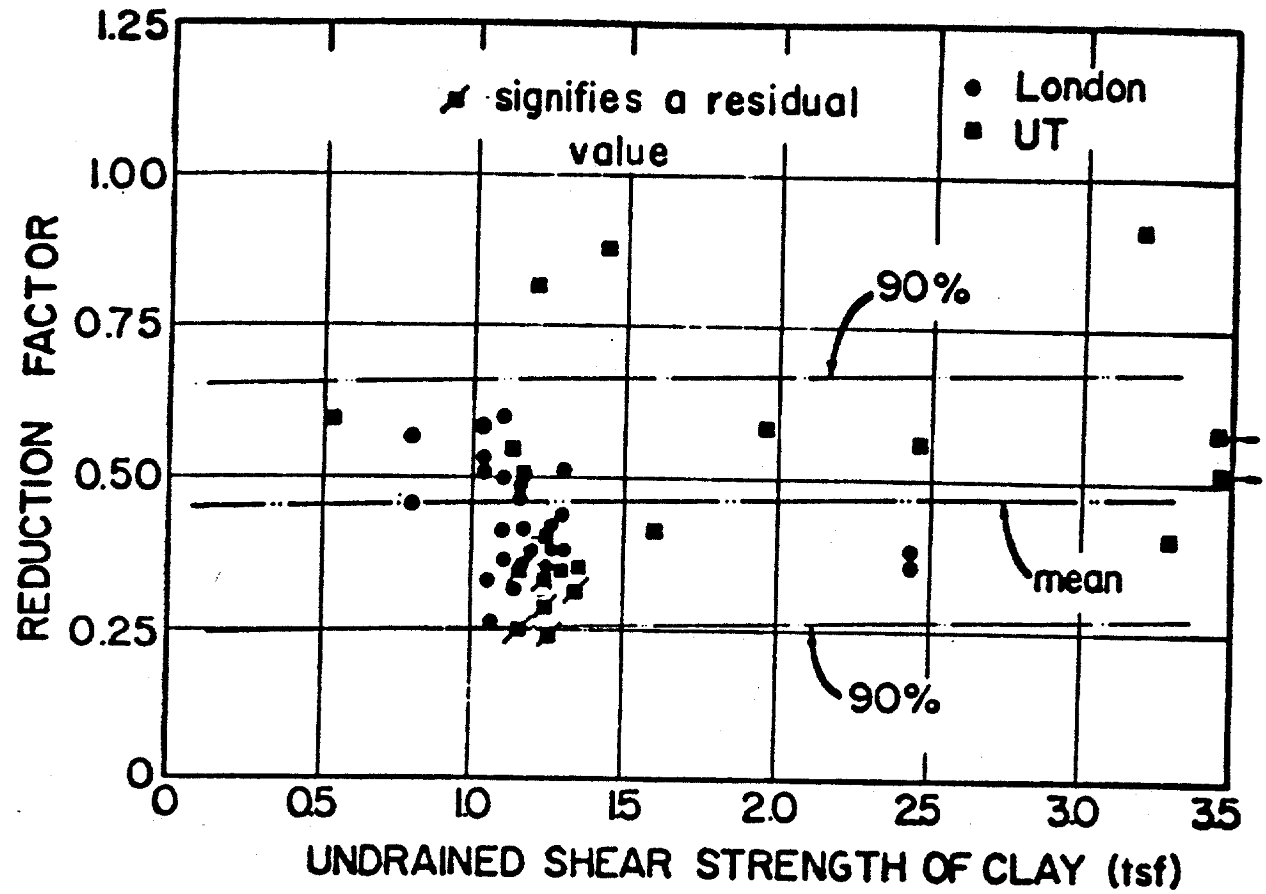
Note: Results of cement-water grouts are affected significantly by a film of water that forms at the steel-grout interface which cannot drain due to test set-up. These results may be too conservative for field cases where drainage can occur. Multiple high pressure grouting can potentially improve steel-grout adhesion by squeezing the free water out.

RESULTS OF LABORATORY STEEL-GROUT BOND STRENGTH
(Adapted from Sol Expert International, 1973)

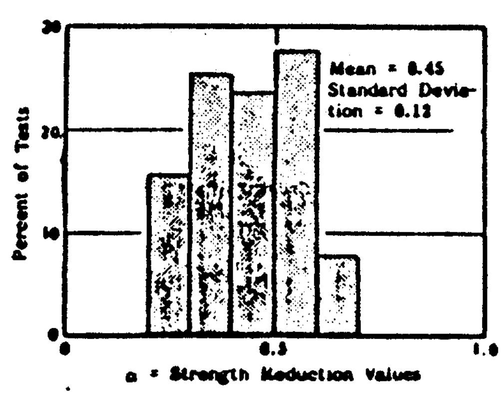




CONCEPTUAL BEHAVIOR FOR DRILLED AND GROUTED PILE COMPOSITE SECTION



a) Reduction Factor (α) for Grouted Piles in Clay
(After Reese and Wright, 1977)

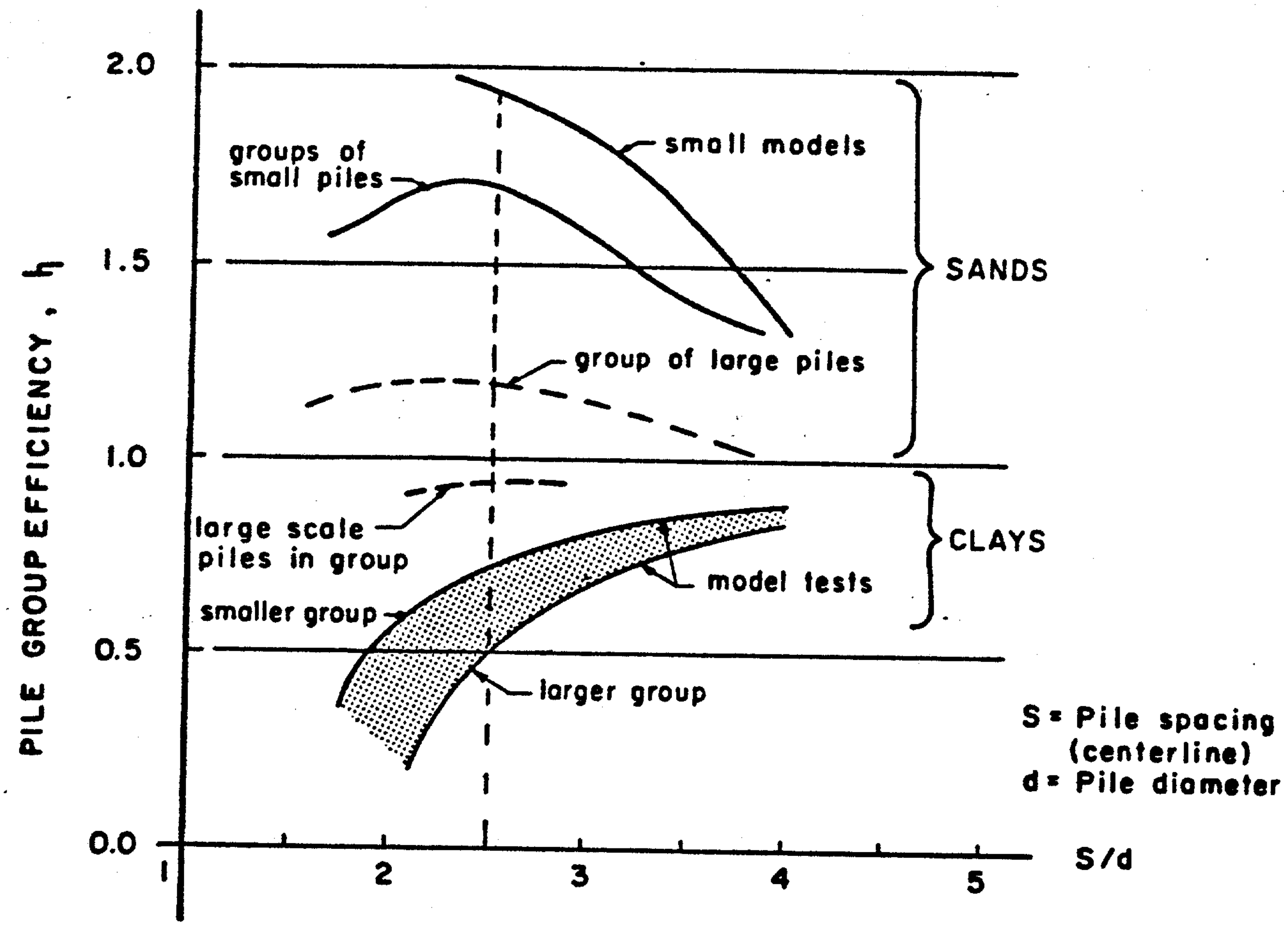


(b) Histogram of α Values
(After Kraft and Lyons, 1974)

REDUCTION FACTOR (α) FOR GROUTED PILES IN CLAY

Source	α Values: Normally Consolidated Clay	α Values: Heavily Overconsolidated Clay	Limiting Adhesion Ksf
Kraft and Lyons (1974)	0.5 to 1	0.3 to 0.5	2.0 to 4.0
Engeling and Reese (1974) Quiros and Reese (1977)		0.3 to 0.6	1.0 to 4.0
Reese et al (1976)		0.3 to 0.5	0.8 to 1.8
Reese and Wright (1977)		0.3 to 0.5	
Skempton (1959)		0.45	Limiting skin friction value = 2.0 ksf
API (1986)	Should not exceed those recommended for driven piles	May exceed values given for driven piles	Value should be determined by some reliable method based on the amount of soil disturbance

LITERATURE DATA - RECOMMENDED α VALUES FOR DRILLED AND GROUTED PILES



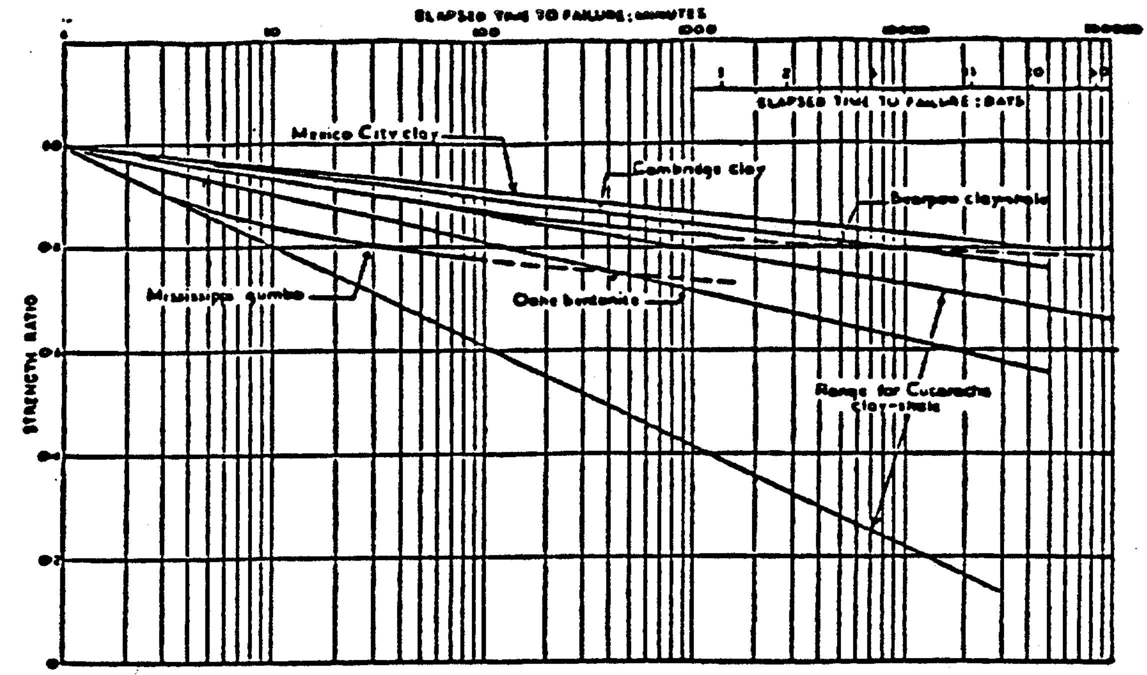
QUALITATIVE ANALYSIS OF PILE GROUP BEARING CAPACITY
(After Vesić, 1975)

Reference	Test Location	Clay Type	Pile Diameter in.	(1) Degraded Shear Transfer after Cyclic Loading	(2) Degradation Threshold
Holmquist and Matlock (1976)	Laboratory	Remolded Highly Plastic	1	33%	75%
Bogard and Matlock (1979)	Laboratory	Remolded Highly Plastic	1	40 - 80%	-
Grosch and Reese (1980)	Sabine, TX	Highly Plastic	1	39 - 46%	-
McAnoy et al (1981)	Cowden, England	Silty	7.6	-	80% (3)
Kraft et al (1981)	Empire, LA	Highly Plastic	14	-	80% (3)
Poulos (1981)	Laboratory	Remolded Highly Plastic	0.8	(3&4) 50 - 70%	(3) 70 - 80%
Karlsrud and Haugen (1983)	Haga, Norway	Overconsolidated	6	42% (4)	91%
Doyle and Pelletier (1985)	Long Beach, CA	Silty	30	61 - 85%	-
Briaud and Felio (1986)	Laboratory	Kaolin, Mississippi Delta Sedim.	0.75	65 - 89%	58 - 80%
			0.75	76 - 89%	80%

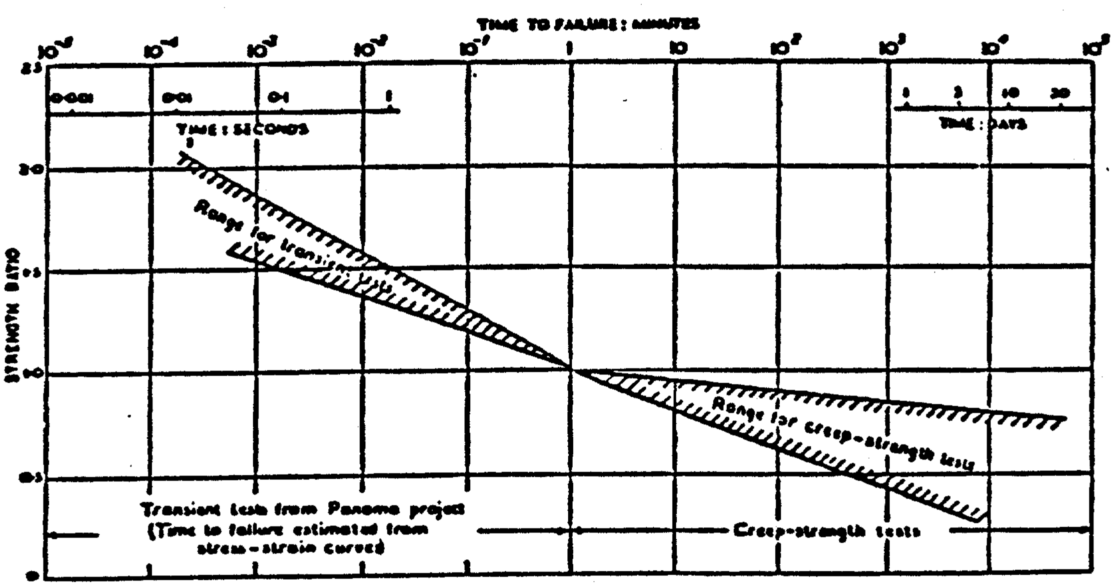
Notes:
(1) Values expressed as percentage of ultimate static resistance.
(2) Cyclic stress level above which large pile displacement occurs, expressed as percentage of ultimate static resistance.
(3) Based on pile-head load-deformation relationship.
(4) Degradation also recorded for one-way cyclic test.

DEGRADED RESISTANCE AND CYCLIC THRESHOLD FOR SELECTED PILE TESTS

The Earth Technology Corporation

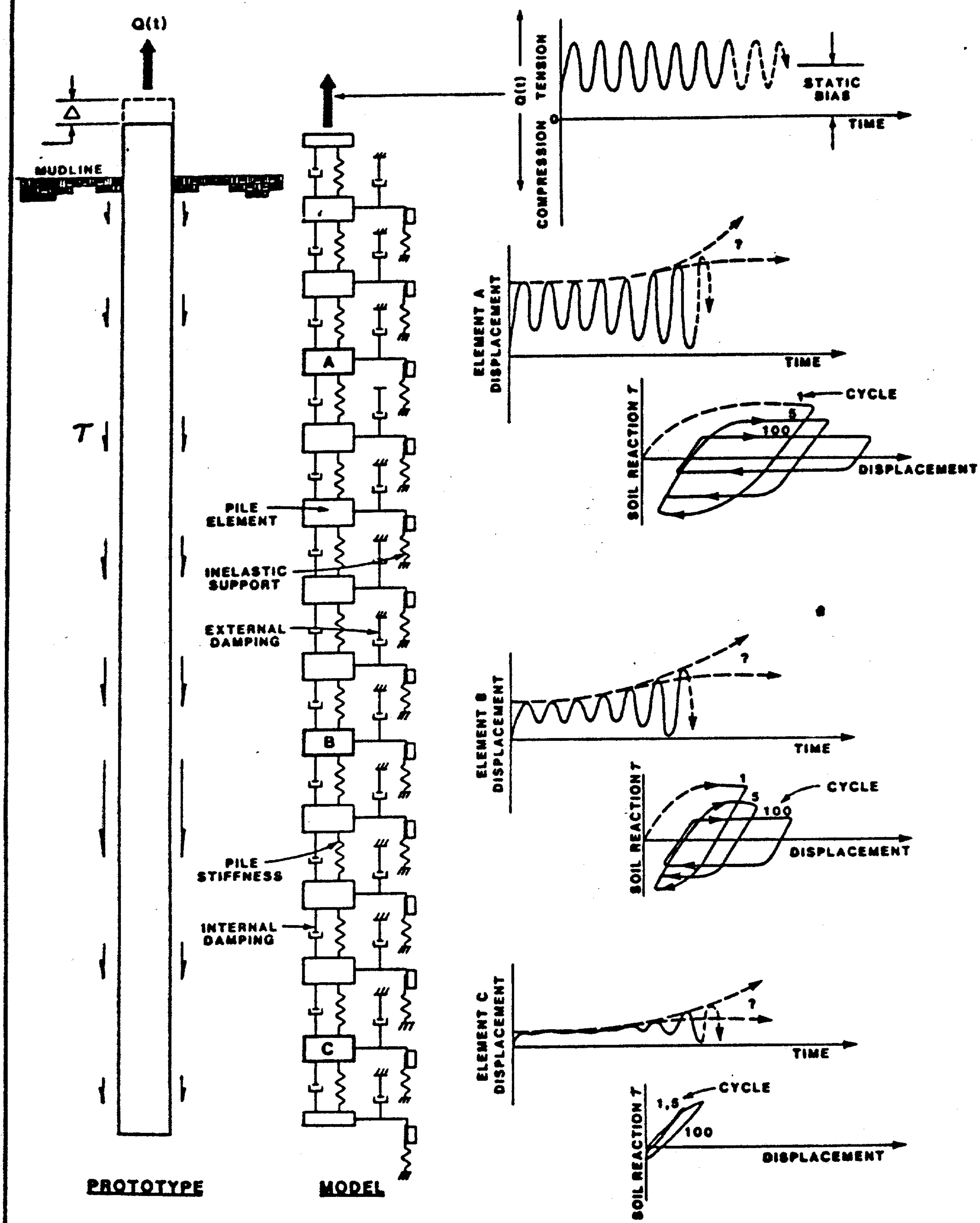


A) LOADING RATE EFFECTS FOR VARIOUS SOILS

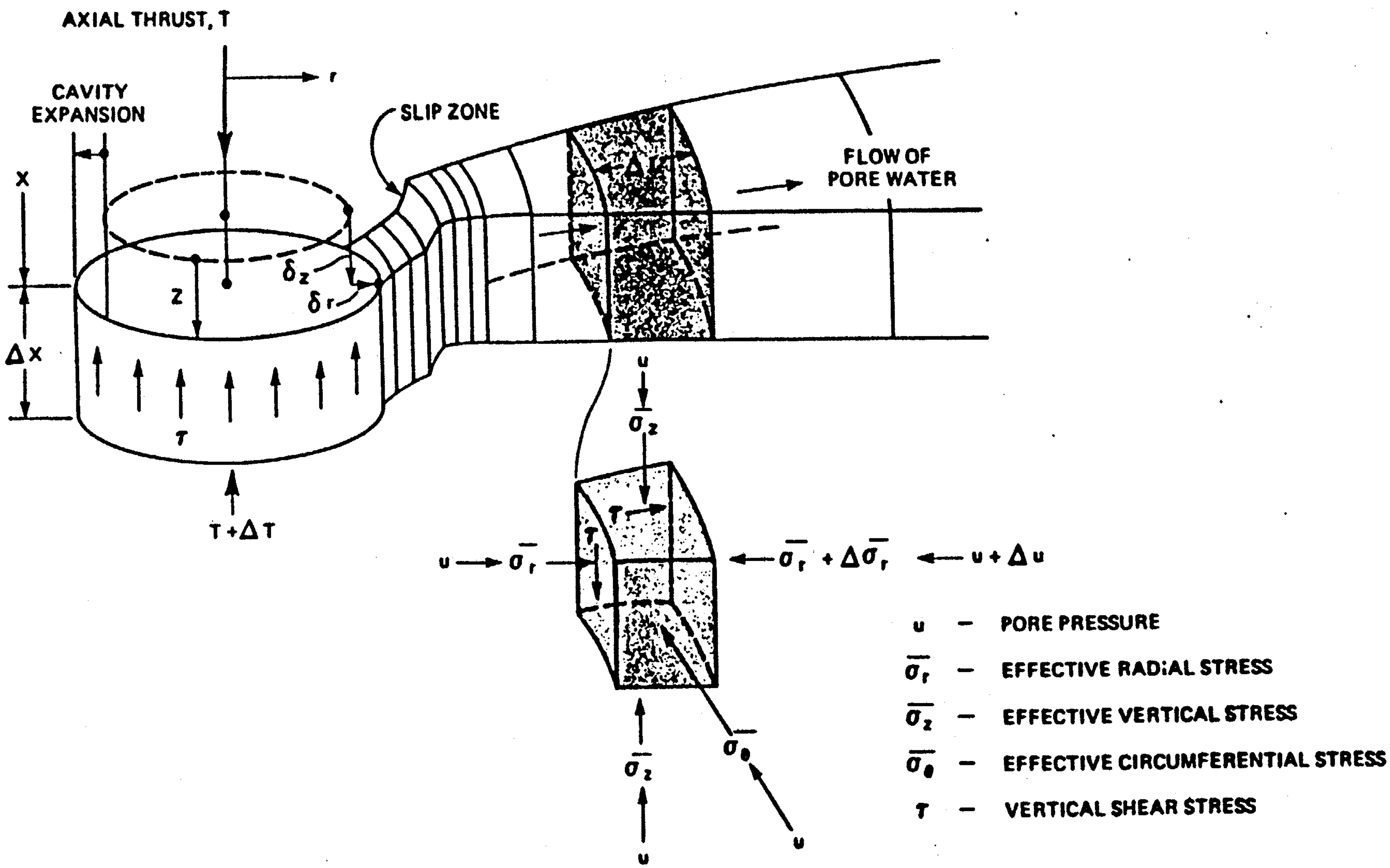


B) LOADING RATE EFFECTS FOR CUCARACHA CLAY-SHALE

LOADING RATE EFFECTS
(After Casagrande and Wilson, 1951)



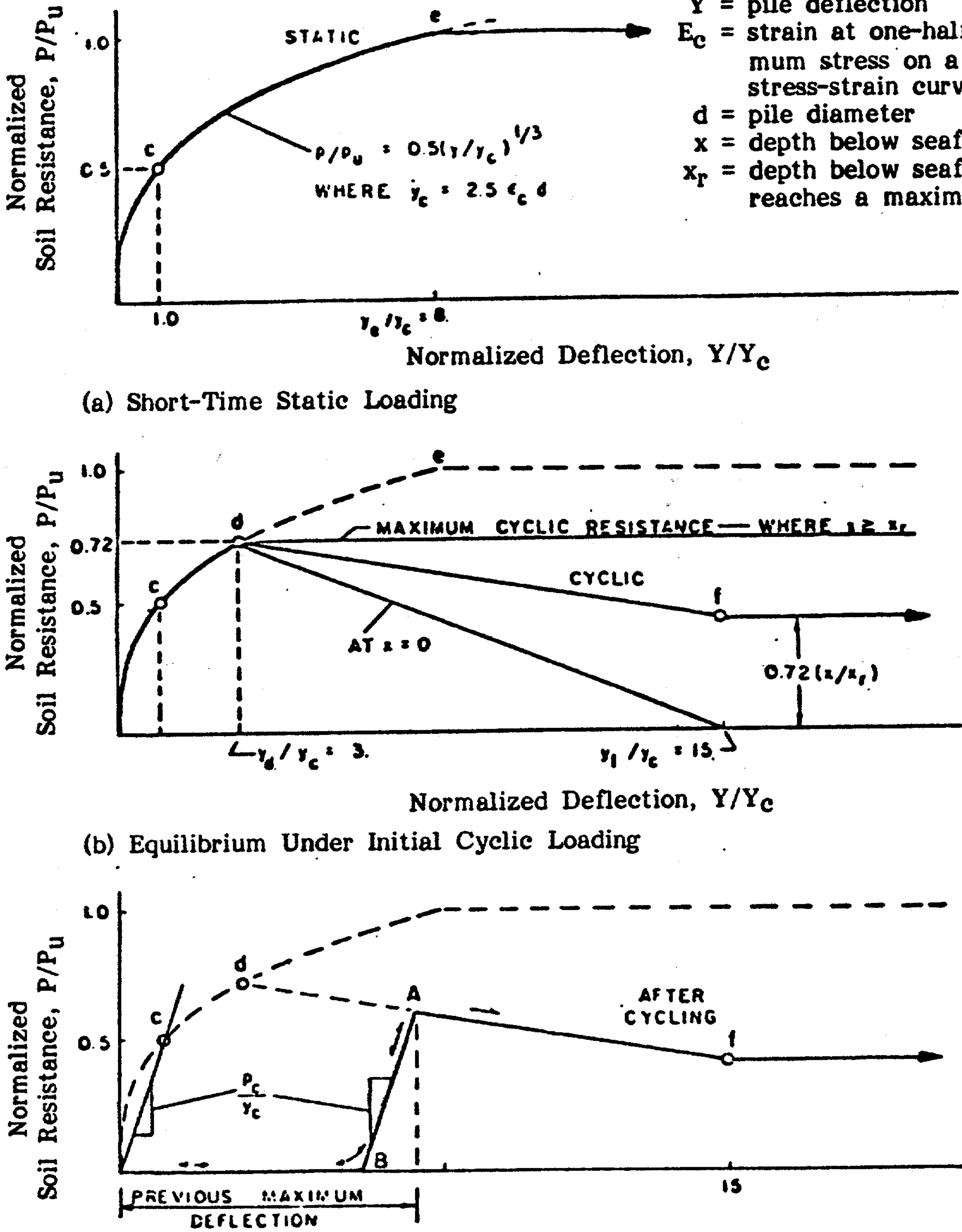
DRIVE MODEL AND PROGRESSIVE ADJUSTMENTS UNDER CYCLIC LOADING



SINGLE-SLICE MODEL FOR PILE INSTALLATION, CONSOLIDATION AND LOADING SIMULATION USED IN THE CASH PROGRAM

Definition of Terms

P = soil resistance
P_u = ultimate soil resistance
Y = pile deflection
E_c = strain at one-half the maximum stress on a laboratory stress-strain curve
d = pile diameter
x = depth below seafloor
x_r = depth below seafloor where P_u reaches a maximum

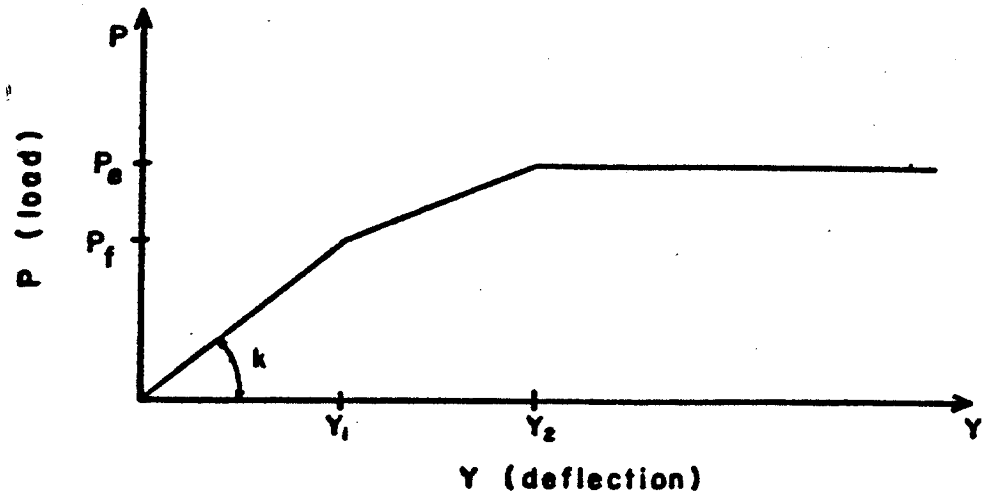


(a) Short-Time Static Loading

(b) Equilibrium Under Initial Cyclic Loading

(c) Reloading After Cycling

SOFT CLAY CRITERIA FOR CONSTRUCTING P-Y CURVES
(After Matlock, 1970)



Step 1) Compute k as:

$$1/_k = \frac{1.33}{3\,E_p} R_o \left(\frac{R}{R_o} \times 2.65\right)^{\alpha} + \frac{\alpha}{3\,E_p} R$$

where E_p = Pressuremeter modulous

R_o = 30 cm

R = Pile radius

α = Empirical constant

= 0.33 to 0.5 for sands

= 0.5 to 0.67 for silts

= 0.67 to 1.0 for clays

Step 2) Compute y_1

$$y_1 = {}^{p_f}/_k$$

where p_f = yield pressure as determined from pressuremeter tests

p_f = 2 RK

Step 2) Compute y_2

$$y_2 = y_1 + 2\,(p_l - p_f)\,/k$$

where p_l = limiting pressure as determined from pressuremeter tests.

MENARD P-Y CURVE FORMULATION